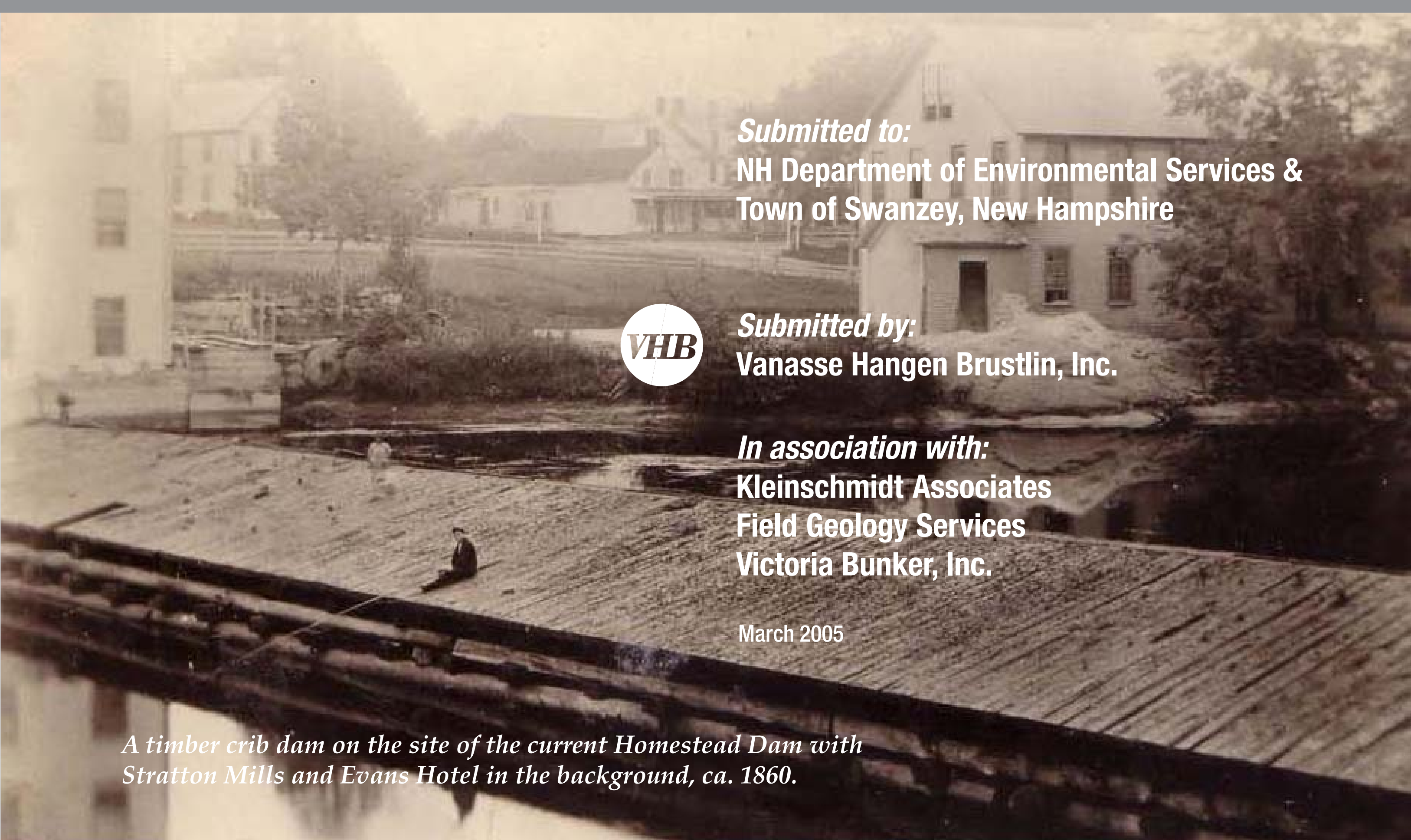
FEASIBILITY STUDY

Feasibility Study

Homestead Woolen Mills Dam
West Swanzey, New Hampshire





Submitted to:
NH Department of Environmental Services &
Town of Swanzey, New Hampshire

Submitted by:
Vanasse Hangen Brustlin, Inc.

In association with:
Kleinschmidt Associates
Field Geology Services
Victoria Bunker, Inc.

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A timber crib dam on the site of the current Homestead Dam with Stratton Mills and Evans Hotel in the background, ca. 1860.

Technical Report

Homestead Dam Feasibility Study

West Swanzey
New Hampshire

Prepared for **NH Department of Environmental Services & Town of Swanzey, NH**

Prepared by ***VHB*/Vanasse Hangen Brustlin, Inc.**
Bedford, New Hampshire

In association with **Kleinschmidt Associates**
Field Geology Services
Victoria Bunker, Inc.

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1 Background

1.1 Introduction

The fate of the Homestead Woolen Mills Dam has been under consideration for several years, prompted by the fact that the structure is unsound and in danger of failure. Following a 1997 inspection of the dam which found several deficiencies, the owner of the dam contacted state officials to learn about the process for removing the dam, which no longer performed any function in the operation of its associated mill and had become an economic and safety liability. State officials responded that its removal would be acceptable and would in fact benefit restoration of fish habitat in the Ashuelot River, which has been a focus of the NH Fish and Game Department, the US Fish and Wildlife Service and the NH Department of Environmental Services. Already, two dams on the lower Ashuelot in Hinsdale and Winchester have been removed, which has improved fish habitat and removed barriers to migrating anadromous fish. These removals have also helped to restore the river to a free flowing condition, which has a variety of environmental benefits.

However, several issues have complicated a decision on the fate of the dam. While a great deal of analysis during the past several years has looked at components of the Homestead Dam, a synthesis of the information and issues associated with the dam is needed. The scope of these issues is well known, and a comprehensive understanding of potential impacts is needed to ensure that public concerns are addressed and that stakeholders have all of the information they need to make an informed decision.

Among the major issues are concerns for the preservation of the Thompson Covered Bridge, an important crossing of the Ashuelot River that is listed on the National Register of Historic Places. Swanzey citizens have also expressed concerns about the impact that elimination of the impoundment would have on nearby wells, on floodplain plant communities upriver, on water supplies for fire fighting, and on recreational opportunities in the community. The Division of Historical Resources is concerned about adverse effects on the potential historic district in the vicinity of the dam. And, while much of the discussion has centered on the potential removal of the Homestead Dam, no substantial information has been presented on potential alternatives to removal.

In response to community concerns, the NH Department of Environmental Services (DES) in cooperation with the National Oceanic and Atmospheric Administration (NOAA), the Town of Swanzey, the US Fish and Wildlife Service (USFWS) and the NH Fish and Game Department (NHF&G), commissioned this Feasibility Study to develop information on the costs, impacts, and benefits of the removal of the Homestead Dam and compare this information to alternatives such as repairing the dam while adding provisions for upstream and downstream fish passage. The Study takes advantage of the existing available information. Where refinement or new studies were needed, the Study presents and explains these new data.

To ensure public input during the Feasibility Study phase, the Town and DES formed an Advisory Group made up of local residents and other interested parties. The Advisory Group is a forum for providing input to the study team, and helps to provide an additional conduit for the distribution of study information to the community and each member's constituency. The Advisory Group is not a decision-making body, but helps to review and comment on study materials, and advises the study team in guiding the development of the project. In addition to the Advisory Group, a Public Informational Meeting was held in May 2004 to discuss the scope of the study and to solicit opinions and information from interested citizens. Additional Advisory Group and Public Informational meetings are planned to be held after release of the Feasibility Study to discuss and refine the analysis. Through this community involvement, it is hoped that the study results will be fully understood and a sense of ownership in the process will develop that will enhance the likelihood of successful implementation of the selected alternative.

1.2 Project Purpose and Goals

During initial stages of this study, three primary goals were identified by the DES and the community. Specifically, the selected alternative must achieve the following:

1. Attain dam safety, whether through dam repair or removal,
2. Provide fish passage, whether through dam removal or installation of effective fish passage, and
3. Ensure the stability of the Thompson Covered Bridge, regardless of the project outcome.

Thus, each alternative will be evaluated to see if it provides for each of these three goals. Additionally, this study will provide a better understanding not only of these goals, but also ecological outcomes, structural engineering constraints, fluvial geomorphology, socio-economics, historic and cultural resources, and other issues. The intent of the feasibility study is to review the significant amount of existing information, gather additional necessary information and synthesize these resources to identify and analyze a range of alternatives. The preferred alternative will meet all project goals, be feasible to conduct, and, ultimately, be supported by the dam owner, DES, and the community.

1.3 The Ashuelot River and its Watershed

The Homestead Dam is located on the Ashuelot River in West Swanzey, New Hampshire. (See **Figure 1.3-1**) The river (and its watershed) contains significant natural resources, and is arguably one of the most important rivers in the Connecticut River basin. This is made evident by the fact that the Ashuelot was nominated as a "designated river" under NH Statute RSA 483:10 by the communities through which it flows. The Legislature approved this nomination in June 1993, which afforded the river special protection through the New Hampshire Rivers Management and Protection Program. Through this program, a river-wide management plan was formulated and adopted by the river's "Local Advisory Committee" in December 2001 (ARLAC 2001).

The Ashuelot River watershed encompasses nearly 420 square miles. The river flows approximately 64 miles from its headwaters at Pillsbury State Park in the town of Washington to its confluence with the Connecticut River in Hinsdale (**Figure 1.3-2**). Over its first 30 miles, the river drops quickly at a rate of 37 feet per mile. Slopes decrease rapidly in the floodplains of Surry, Keene and Swanzey. The river has a particularly low gradient through the towns of Keene, Swanzey and Winchester. The gradient from the Colony Mill dam in Keene to the Homestead Dam, as estimated from US Geological Survey (USGS) topographic maps, is approximately 12 feet over 8.7 river miles. This is an average of 1.4 feet per mile, which is quite flat, especially when compared to upper portions of the watershed. A similar gradient exists for the next 7 or 8 miles below the dam into Winchester, where the river gradient becomes steeper as it turns west to follow a narrow valley to the Connecticut.

As will be discussed in more detail in Chapter 3, the Ashuelot River and its watershed contains areas with both cultural and environmental significance. The river boasts the site of the oldest known evidence of man in New Hampshire, dating back 10,500 years. Evidence of this pre-historic past is contained within the river itself – a fishing weir built by pre-contact Native Americans can be found approximately three miles upstream of the Homestead Dam (Bunker *et al.* 2004). The watershed has also been identified by the US Fish and Wildlife Service (USFWS) as one of the four most important refuges for the federally-listed endangered dwarf wedge mussel *(Alasmidonta heterodon)*. It is considered a vital tributary of the Connecticut River in support of restoring the once abundant runs of American shad *(Alosa sapidissima)*, Atlantic salmon *(Salmo salar)* and other anadromous fish species (Connecticut River Atlantic Salmon Commission 2003; Sprankle 1998). The Ashuelot is fed by dozens of tributary streams, many of which are in outstanding condition and support healthy populations of brook trout and other coldwater fish.

1.4 Description of the Homestead Dam

The Homestead Woolen Mills Dam, also referred to as the Homestead Dam, West Swanzey Dam, Swanzey Dam, and the Dickinson Dam, is a timber crib dam approximately 167 feet long by approximately 12 feet high. The Homestead Dam is a non-gated or "run of the river" dam. It provides minimal flood control protection, as the design permits the water to flow over the top of the spillway rather than providing for freeboard to store water. The superstructure consists of 12 inch to 18 inch diameter logs which are spiked, bolted or pegged together in successively narrower squares, *i.e.,* "crib" layers, stacked to form an inverted pyramid. The crib work is covered with timber deck planks, which are approximately 8 inches wide by 3 inches thick, to create the spillway. (See **Figure 1.4-1** for a plan of the mill and dam surroundings as well as a partial cross-section drawing of the dam.)

The crib work of the Homestead Dam currently contains only minimal rock ballast. The original structure was likely stone-filled, but during the many repairs made to the dam, the ballast may have been removed and not replaced. Ralph and Todd Osgood, the contractors who last repaired the dam in 1992, were unaware of any posts or devices anchoring the dam to the substrate (Wood 1999). The weight of gravel and rock on the upstream slope of the structure apparently helps to hold it in place. Much of the rock and gavel on the upstream slope of the dam is part of a coffer dam which was placed in the river to aid in heavy equipment access during dam repair. There are no records documenting the repair history of the dam prior to the 1992 repairs.

The dam complex also includes a 700 foot long, 40 foot wide tailrace on river left (east) which evidently once provided water to a box mill that appears in historic photographs of the area. A still-standing concrete sluice on this end of the dam is further evidence of this past function. The upper portion of the tailrace has been filled, in part with the demolition debris of the former box mill, but the lower portions of the tail race are intact.

The original construction date of a dam at the present location is somewhat unclear, but could be as early as the late 1700s. The New Hampshire Dam Safety program lists the original construction date of the present structure as 1910. However, it is known that a dam existed in this location prior to that date, and a *circa* 1860 photograph shows a timber crib dam of the same basic shape as the existing structure (See Photograph on page 79 of the West Swanzey Village Historic District Area Form, **Appendix A**). Historic records indicate that a dam was in place on the Ashuelot River at West Swanzey even earlier than 1860 - in the late 18th century - though it is not clear if that dam was in the same location as the present Homestead Dam. In fact, a number of dams existed on the Ashuelot during this period, and their impact on fisheries is made evident by the fact that the New Hampshire General Assembly in 1789 mandated that sluice gates at dams at Hinsdale, Winchester, Swanzey, and Keene be kept open between May and July for fish passage (Read 1892).

The dam once provided water for the Homestead Woolen Mill (and apparently its predecessor, the West Swanzey Manufacturing Company) as well as the box mill that was located on the east side of the river near the still-existing tailrace. However, with the destruction of the box mill by fire in the 1960s and the closing of the woolen mill in 1985, the dam no longer served any useful function.

The brick Homestead Mill is still located on the west side of the river, and although it is not currently being used as a woolen mill, it now houses a mixture of light industrial, commercial, and storage businesses. The former Homestead Mill water intake between the mill and upstream covered bridge has been abandoned, and the dam's impoundment is no longer used by any of the mill's occupants.

1.5 Engineering Evaluation of the Dam

A full inspection and engineering analysis of the existing dam was performed to determine the existing condition of the Homestead Dam. This includes analysis of the existing dam's, structural safety factors, and remaining service life. **Appendix B** contains the inspection report issued by Tom Kahl, PE of Kleinschmidt Associates.

It should be noted that previous inspections of the Homestead Dam by the DES Dam Bureau in 1997 found several deficiencies in the existing dam structure which are the subject of an outstanding Letter of Deficiency. In August 2004, the consultant team visited the dam site to complete the first formal inspection of the dam since 1997. As could be predicted, the condition of the dam has continued to deteriorate.

The dam was inspected on August 10, 2004 by engineers and scientists from Kleinschmidt Associates (KA) and Vanasse Hangen Brustlin (VHB). The upstream water level was approximately 2 feet below the crest of the dam, which allowed the personnel convenient access across the entire dam crest. During this visit, the downstream toe of the dam was inspected by boat, both abutments were inspected, and dimensions of the existing structure were recorded. Mr. Doug Brown, owner of the dam and the Homestead Mill building was also interviewed. A number of photographs are shown in the Kleinschmidt Associates inspection report in support of additional descriptions of the dam below.

1.5.1 Headpond and Abutments

Photograph No. 1 (Attachment B of Kleinschmidt's Report) is looking from the left towards the right hand abutment and shows the gravel bar upstream of about 120 ft. of the left hand side of the dam that was used as a cofferdam and access road for the 1992 repairs. It is interesting to note in this photograph that the water level upstream of the cofferdam is higher than the water level between the cofferdam and timber crib dam. This means that the cofferdam is less permeable than the timber crib dam and acts as a hydraulic control. The current brick Homestead mill building on the right hand shore can also be seen in the background. Photographs No. 2 and No. 3 show in detail the mill foundation immediately upstream and downstream respectively of the spillway that comprises the dam's right hand abutment. Photograph No. 4 is looking towards the dam's left hand abutment, and the left hand abutment is shown more closely in Photograph No. 5. The left hand abutment has a concrete foundation for an abandoned hydro-mechanical water wheel. Photograph No. 6 shows the silted in intake on the upstream side of the dam, and Photograph No. 7 shows a 4 ft. wide by 5 ft. high opening from the abandoned turbine flume that is currently covered by a steel plate. Photograph No. 8 shows concrete erosion of the left hand abutment wall between the crest and abandoned intake.

1.5.2 Dam Foundation

Appendix C contains the 2002 geotechnical core borings at two locations along the dam and river bottom profile that show that the dam (bottom at Elevation 443 to 445) is founded on widely graded sand with silt and gravel, which Boring B-7 designates as glacial till. Bedrock was not encountered in either borings B-1 or B-7, which penetrated 21 and 25 feet respectively below the dam crest. Similarly, bedrock was not encountered in any boring (n=7) in the immediate vicinity. The dam timbers are probably not pinned into the underlying soil in any way, such as with piles. There is also no evidence that the dam has cutoff trenches or a sheet pile cutoff underneath the structure, which is a common feature of timber crib dams to prevent scour underneath the dam and a resulting breach.

1.5.3 Dam Geometry and Crest Condition

The overall crest and dam geometry is depicted in **Figure 1.4-1**. Although the upstream face of the dam below the water level could not be observed, photographs from the 1992 repairs included in **Appendix B** confirm that the upstream face maintains a constant slope to the upstream heel of the dam (where the dam intersects the upstream river bottom). As reflected in Photographs No. 9, 10, and 11, the timber planking on the dam crest was severely decayed. A pocket knife easily penetrated the timber planking ½ inch to 1 inch, which is significantly more than the ¼ inch penetration normally regarded as indicating a decayed timber. This means that the planking has only nominal remaining structural strength. As shown in the site visit photographs, the planking has numerous holes and gaps, and Photograph No. 12 shows a leakage induced headpond whirlpool near the left hand abutment. Photograph No. 13 shows the large approximately 2 foot settlement of the downstream crest planking located about 40 feet from the right hand abutment, and a similar settlement was observed about 15 feet from this abutment.

As shown in Photograph No. 14 the downstream vertical face of the dam is uncovered, and allowed visual observation into the dam. There was only occasional and minimal rock ballast scattered along the cribbing floor. The 1992 repair photographs also do not show any rock ballast in the timber cribbing. Photograph No. 15 shows the location of a missing horizontal cribbing timber along the 70 linear feet of spillway closest to the right hand abutment. Note that this is also the location where the crest settlement in Photograph No. 12 occurs. The timber cribbing accessible along the downstream face showed significant wood decay.

1.5.4 Maintenance History

Discussions with Mr. Doug Brown, the owner of the Homestead Mill and Dam, confirmed that the last major rehabilitation replacing a large number of planks had been around 1992. Mr. Brown indicated that typically some plank replacement/repair would need to be performed every few years. But, since the dam was not performing any function for the mill, no repairs had been performed for several years. Mr. Brown recalled that the cribbing timbers had been repaired once in the last 30 years, and the photographs in Attachment D also show some new cribbing. Mr. Brown also said that he did not recall ever seeing rock ballast inside the dam cribbing.

1.5.5 Hazard Classification

The New Hampshire Department of Environmental Services (DES) has given the Homestead Dam a Class A Hazard classification per NH Administrative Rule Env-Wr 101.04. This means that failure of the dam would not result in possible loss of life as defined in Env-Wr 101.29, but would result in any of the following:

- Minimal economic loss;
- Major damage to town and city roads; or
- Minor damage to Class I and II state highways; or
- The release of liquid industrial, agricultural, or commercial wastes or municipal sewage.

1.5.6 Stability Analysis

Based upon the field visit dimensions and gathered information, the stability of the existing timber crib dam was analyzed assuming a normal pond condition with the water level at the top of the spillway crest. As shown in Kleinschmidt's report (**Appendix B**), these calculations utilize a 12 foot deep lateral hydrostatic loading, although no uplift forces were added because the timber crib bottom is open - which permits seepage that would relieve any uplift pressure. The calculated Factor of Safety for the existing dam against sliding is 1.12, which is significantly less than the minimum 2.0 Factor of Safety that is used by the Federal Energy Regulatory Commission (FERC 2003) as a minimum for low hazard dams. The low 1.12 sliding

Safety Factor means that the existing structure does not presently meet a normally accepted minimum Factor of Safety.

Overturning was not considered as a potential mode of stability failure. As mentioned in Craeger and Justin (1948), overturning is not considered a failure mode in crib dams because of their cross sectional width. Also, the timber crib construction does not allow for rotation about the dam toe.

1.5.7 Stability and Strength

The existing dam's calculated sliding stability Factor of Safety of 1.12 is significantly less than the normally required minimum of 2.0. This means that the existing dam does not meet current engineering standards, and that the existing dam has an increased risk of failure. Conceptual calculations in **Appendix B** show that the portion of the dam cribbing downstream of the crest would need to have rock ballast filled to at least a 53 percent void density to provide enough weight to provide a minimum 2.0 Safety Factor against a sliding failure.

The primary structural element of this timber crib dam is the upstream planking that needs to support the weight of the water and accumulated silt. It is critical to the dam's function that this upstream planking remains intact without any holes or damage that will allow leakage. The two layers of upstream planking that were exposed along the dam crest showed decay penetrating 1 inch to 2 inches into the 3 inch thick planks. Therefore, these planks are of dubious structural integrity, and do not offer long term, reliable service lives.

1.5.8 Serviceability – Upstream Water Levels

The extremely poor condition of the existing upstream planking is reflected in the considerable leakage through the existing dam that prevents the dam from maintaining the headpond at the crest of the dam during many ranges of river flows. The high permeability of the existing dam is reflected by the previous comments on Photograph No. 1 where it can be noted that the upstream cofferdam is maintaining a higher water surface than the timber crib dam. According to the USGS (Brian Mrazik, personal communication with Matt Bernier, 2004), the original stream gauge just upstream of the Homestead Dam could not be maintained because the deteriorating condition of the dam caused variability in the stage-discharge relationship. The stream gauge and transducer were moved upstream of the cofferdam to provide more stability in the stage-discharge relationship, although there are still "variability issues" with the new location. Therefore, both the cofferdam and timber dam appear to be deteriorating or eroding and changing water levels upstream of the dam, with the deterioration of the timber dam apparently more significant than the erosion of the cofferdam. Another example of the existing dam's high leakage is that during the August 10, 2004 inspection, even though the headpond was 1.55 feet below the dam crest in the morning, leakage through the dam was greater than the river flow since the water level dropped approximately 4 inches from about 10 AM to 4 PM. Therefore, the existing dam deterioration is presently resulting in a continual average lowering of the headpond and the historical water levels. For example, on August 10, 2004 existing dam leakage had essentially decreased the effective height of the dam by 1.55 feet, which is nearly 13 percent of the total dam height.

1.5.9 Remaining Service Life

Historically timber crib dams are reported to have a 20 to 30 year service life (Craeger and Jensen, 1950 pp 458), although with continual maintenance some timber crib dams may approach 50 to 60 years. Typically, the higher portions of the timber which are more exposed to alternating wet dry cycles and oxygen experience accelerated decay and need to be replaced more often, while wood sections lower in the dam remain competent for much longer periods without replacement. The primary cause of deterioration for a timber crib dam is wood decay that causes settlement of the cribbing and leakage through the upstream wood planking. Wood decay is caused by fungi or bacteria that depend on the proper combination of moisture, temperature, and oxygen. Therefore, wood that is kept dry, below freezing, or submerged where it is inaccessible to oxygen, does not decay. Obviously, except for winter, the wood cribbing and planking of the Homestead Dam will be exposed to all three of these conditions and therefore continued wood decay is inevitable.

The rate of decay and remaining existing service life is impossible to accurately quantify, particularly since a structural/stability failure of the structure is dependant on variable loadings such as seasonal ice/debris and flood water levels. But as previously noted the existing dam is continually deteriorating and presently lowers the headpond below historic water levels.

The most probable failure scenario is that the dam's deterioration will result in a continual decrease in headpond water levels, and then eventually high loads on the dam from seasonal ice/debris or flood water levels will cause a larger (probably the right hand portion of the dam which is in poorer condition and also experiences the river's main channel flow) section of the dam cribbing to dislodge and suddenly rupture a large area of upstream planking. Although it is impossible to predict when this would occur, based upon historical experience and the deteriorated condition of the existing dam, it is estimated that unless the existing dam is repaired or replaced it will loose most of its water retention ability within the next five to ten years, perhaps sooner with a high flow event and scouring of the cofferdam.

2

Alternatives

2.1 Introduction

One key element of the feasibility study is to define a reasonable range of alternatives for consideration by the stakeholders. Based on discussions with the resource agencies, the Advisory Group and the general public, the following conceptual alternatives were developed for discussion in the Feasibility Study. The study provides a discussion of the costs associated with each of these alternatives (See also **Appendix D),** and later chapters provide an assessment of the impacts and benefits of each of these alternatives.

2.2 Alternative A - No Action

This alternative would be defined as "no repair or restoration work on the Homestead Dam." Under this scenario, the existing dam would remain as is, with no repair or maintenance. The condition of the dam would certainly continue to deteriorate, and safety and liability concerns would become even more acute. Additionally, due to additional and increased leaking of the dam, the impoundment level is expected to fall. Inevitably, the dam would fail, resulting in unpredictable damage to property.

As explained in Chapter 1, it is readily apparent that this alternative is not feasible due primarily to safety issues, based on the 2004 inspection, a review of dam inspection reports and on a general knowledge of the Ashuelot River. Nevertheless, its inclusion in the study is useful to provide a baseline against which to evaluate other alternatives. Obviously, there are no direct economic costs associated with this alternative.

2.3 Alternative B - Full Dam Removal

This alternative involves the physical removal of the entire existing dam structure, the associated cofferdam, and subsequent reshaping of the river channel upstream and immediately downstream of the dam. (See Figure **2.3-1.**) While the full removal alternative will typically provide the most ecological and water quality restoration benefit, it will also create the most substantial change in the headpond elevations and river hydraulics. These changes, in turn, may consequently have effects on the Thompson Covered Bridge, on wetlands and floodplain communities along the impoundment, potential impacts to wells along the river, *etc*. These potential impacts are discussed in more detail in Chapter 3 while the direct economic costs associated with the construction work are presented in Section 2.7 below.

Removal of the dam is expected to take approximately ten days to two weeks of work, and would occur under environmental controls designed to limit any temporary environmental effects. Such work would only occur during the low flow months of August and September. Equipment would be staged on the east bank of the river, near the USGS gauging station, where equipment has previously accessed the river. A portion of the bank would need to be graded to allow equipment access, and the existing causeway would be raised and leveled. Removal of the timber decking and interior crib work would use standard construction equipment such as a track excavator. All of the timber decking and cribbing would be removed from the river, working from west to east, and hauled for disposal at a landfill. It is assumed that any stone ballast remaining within the dam could be dispersed on the stream bed. However, the causeway would be removed entirely because it is constructed of non-native fill and needs to be removed to create a stable stream bed. The concrete abutments on either end of the dam would remain in place to minimize construction costs.

Because removal of the dam would result in an unstable riverbed in the immediate vicinity, dam removal will also require reshaping the channel. Preliminary engineering indicates that a reconstructed channel would start at the present base of the dam and would slope upward at a 4 percent grade for a distance of 100 feet (*i.e.*, from elevation 444 feet to elevation 448 feet. This approximately matches the present bathymetry at the channel end points. The new channel would be roughly trapezoidal, 33 feet wide at the top and up to 4 feet deep (averaging 2 feet deep).

It is interesting to note that removal or modification of the Homestead Dam was previously considered by the Army Corps of Engineers for additional flood relief for the city of Keene. But, due to the low gradient through this portion of the valley and the fact that the Homestead Dam actually has minimal effect on flooding in Keene, it was determined that flood mitigation benefit would be minimal and the plan not acted upon. Additionally, the city of Keene also considered removal of the Homestead Dam in an effort to remedy water quality issues associated with the Keene sewer treatment plant which discharges to the Ashuelot (Stephanie Lindloff, personal communication, 2004).

2.4 Alternative C – Replacement of the Dam with Fish Passage

These alternatives consist of two options, both of which involve reconstruction of the dam in place, but which also take action to add upstream and downstream fish passage. Two potential structural "fishway" options are contemplated under this alternative - a traditional Denil fish ladder or a nature-like bypass channel (see Alternatives C1 and C2 below).

The traditional means of repair is to replace the decayed timber cribbing and planks as was done in 1992. Based on the structural assessment of the existing dam as summarized in Section 1.5, it is assumed that piecemeal repair to the dam is not acceptable. Rather, the structural analysis makes clear that the present crib dam does not meet current, accepted safety criteria as recommended by DES and FERC. The most reasonable and economical way to meet these criteria will be to completely replace the structure. That is, these alternatives involve disassembly of the Homestead Dam and subsequent construction of the new dam in its place.

Timber crib dams were economically attractive in the 19th and very early 20th centuries when labor costs were low and wood material was inexpensive, so it was acceptable to rebuild major portions of such a dam every 20 to 30 years. But, because of higher labor and maintenance costs today, timber crib dams are generally replaced by concrete gravity dams that offer service lives of 50 to 100 years without significant maintenance.

However, the NH Division of Historical Resources, as well as some Swanzey citizens, have indicated that the existing dam is a historical resource that is important to the community. Therefore, given the fact that it should be possible to safely replace the existing dam *in kind*, this study assumes that any replacement dam would be of traditional timber-crib construction.

While a new timber crib dam would on its outside appear to be very similar to the existing dam, it would in fact be substantially different in order to provide an adequate margin of safety (as discussed in Section 1.5.7). The new dam would include stone ballast, stream-bed anchoring, and other features to create a new structure that provides an acceptable factor of safety.

Both Alternative C options also include installation of a fishway to the dam. Fishways (also fish ladders or fish passes) are structures placed on or around dams to assist the natural migration of diadromous fish (*i.e.,* fish that move between saltwater and freshwater habitats). Most fishways enable fish to pass around the barrier by swimming and leaping up a series of relatively low steps (hence the term "ladder") into the waters on the other side. The velocity of water falling over the fishway has to be great enough to attract the fish, but cannot be so great as to exceed the target species swimming abilities or to exhaust them to the point where they cannot continue their journey upriver. A discussion of these issues is included in Section 3.7.1.

2.4.1 Alternative C1 - Denil Fishway on River Left

This alternative would involve replacement of the dam along with design and construction of a traditional Denil fish ladder along the eastern side of the existing dam (*i.e.*, river left). Initial review of the dam site suggests that construction of the ladder on river left is far more feasible than on river right due to construction access and the constraint posed by the mill structures on river right.

A Denil fishway consists of a steep flume with a series of internal baffles that are fixed to the floor and walls of the structure. These baffles, in shapes of varying complexity, cause secondary helical currents that ensure an extremely efficient dissipation of energy in the flow by intense transfer of the momentum. The closely spaced baffles create turbulence and thus dissipate the energy of the water passing down the flume to velocities that permit fish movement. In fact, the design of the Denil baffles is so efficient at dissipating the energy of fast flowing water, that this type of fishway is capable of passing more volume, as compared to other types of structural fishways, with the same cross-sectional area. This translates into having a better attraction flow at the entrance of the fishway. Denil fishways can be built with slopes ranging from 10 to 20 percent and are typically constructed of concrete, steel or aluminum. With the design allowing for a steeper slope, this fishway can be constructed in a shorter area as compared to other fishways. Nevertheless, given that the dam crest is approximately 12 feet above the downstream river bed, a Denil ladder at the Homestead Dam would be approximately 150 feet in length.

Generally, Denil fish passes are used for fish larger than around 10 inches. They can be used for smaller species, but only if the size of the baffles or slope are reduced. Thus, Denil ladders, like other structural fishways, tend to be relatively selective at providing passage.

A conceptual design for a Denil fishway at the Homestead Dam includes a concrete 12 foot entrance pool and 12 foot exit pool, two ladder runs rising 6 to 9 feet each, and a center resting pool 12 feet in length. Total length of the passage is approximately 130 feet with a depth of 6 feet. The ladder would have baffle inserts every 2.5 feet of ladder run. The intake would be controlled by an automatic gate tied to headpond elevation. The likely location of the Denil ladder, as well as a conceptual design is shown in **Figure 2.4-1.**

2.4.2 Alternative C2 – Nature-like Fish Bypass Channel

This alternative would involve replacement of the existing dam with a new timber-crib structure and construction of a "nature-like bypass channel" approximately parallel to the river with entrance and exit pools located downstream and upstream of the dam.

As observed by Parasiewicz (2002), traditional structural fishways, such as the Denil ladder, have regular geometries and are consequently more predictable in their engineering design and behavior due to the fact the hydraulics of regular shapes are more precisely understood than irregular surfaces. However, the wide diversity of species and life stages found in riverine ecosystems depend on the variety of flow conditions in complex channels. Consequently, the performance of traditional structural fishways can create conditions that selectively limit the passage of fish species and life stages.

An alternative is to create a fishway that more closely mimics conditions found in natural systems – the "nature-like fishway." A nature-like bypass channel resembles a side channel or tributary of the main river system in function and structure. This type of design increases the potential efficiency of passage for a wider variety of fish and macro-invertebrates, as well as providing habitat and structure (Parasiewicz *et al.* 1998). While this approach is relatively new compared to traditional technical fishways (approximately 30 years as compared to almost 100 years), Wildman *et al.* (2004) have cataloged successful nature-like passages on numerous rivers in North America, Europe, New Zealand, Australia, and Japan.

Bypass channels are typically characterized by a very low gradient, generally 1 to 5 percent, even less in lowland rivers. Rather than in distinct and systematically distributed drops as in pool type passes, the energy is dissipated through a series of riffles or cascades positioned more or less regularly as in natural water courses (Gebler 1998). The main disadvantage of this solution is that it needs considerable space in the vicinity of the obstacle and cannot be adapted to significant variation in upstream level without special devices (gates, sluices). These control devices may cause hydraulic conditions which make fish passage difficult. As with any other fish pass, it is recommended that the fish entrance to the artificial river be located as close to the obstruction as possible. Given the very low gradient, it is sometimes difficult to position the entrance immediately below the obstruction, which means it must be further downstream. This may restrict their efficiency, and consequently make them less useful for large rivers.

As noted above, the former dam tailrace flows approximately 700 linear feet downstream of the dam on river left. Approximately 150 feet of this tailrace immediately downstream of the dam was filled, apparently following demolition of the former box mill that once stood at this site in the 1960s. Nevertheless, during field review of the dam site with Jim Turek (NOAA Restoration Center) on April 20, 2004, it was determined that use of the former tailrace provides a potential opportunity for construction of a fishway. However, one challenge associated with this type of fishway is to generate sufficient flows at the downstream entrance of the channel to attract migrating fish. The tailrace is relatively wide (40 feet) and long (700 feet). Thus, fairly substantial flows would need to be diverted to the bypass channel to provide adequate depths and volumes needed to properly function as a bypass channel. Additionally, the tailrace re-enters the mainstem relatively far downstream from the dam, which further complicates the generation of attraction flows and would likely decrease the efficiency of the bypass channel. Therefore, it is recommended that a relatively shorter bypass channel on river left (in the vicinity of the tailrace) be constructed.

A conceptual design for a natural fish bypass channel at the Homestead Dam was developed (**Figure 2.4-2)**. The channel would be approximately trapezoidal in cross-section with banks at a 2:1 side slope. Its bottom dimension would vary in width from approximately 8 to 10 feet, and would be approximately 4 feet deep. The designed channel slope is 5 percent, which would create a channel of approximately 250 feet long. The channel would be lined with geotextile fabric and varying sized stone. A manually-operated, wooden stop log weir would control intake flows to the passage. This log weir would require continuous monitoring and operation, especially during spring anadromous fish migrations.

2.5 Alternative D – Full Removal with Rock Ramp

This alternative would involve full removal of the existing timber crib dam and construction of a "rock ramp" in the vicinity of the existing dam. Rock ramp fishways are a form of nature-like fishway and can be constructed in a variety of forms. They are generally low-gradient, like the bypass channel, and can be constructed across the entire channel or only a portion of the channel. Like other fishways, the objective of the rock ramp is to provide a stream bed slope and water velocities that diadromous fish can successfully navigate.

Since rock ramps are a relatively new form of fishway, the nomenclature of such structures is not yet standardized. In the case of the Homestead Dam, this report uses the term to refer to a large, full-channel structure. In fact, the rock ramp contemplated for the Ashuelot would, in fact, be designed to act as a dam in order to maintain existing water surface elevations. In essence, the rock ramp alternative replaces the timber crib dam with a similarly functioning rock structure.

The most logical location for such a ramp would be at the current location of the dam, extending upstream and/or downstream some distance. (See **Figure 2.5-1.**) The construction of a rock ramp in this location could take advantage of the existing dam as a cofferdam during construction and could recycle boulders from the timber crib and existing submerged trap-rock cofferdam/access road.

The conceptual design for the Homestead Dam location would use a driven sheet piling curtain 8 feet above the assumed natural stream bed at the upstream face of the causeway. The slope of the ramp is to be 5 percent with rock weirs placed approximately every 20 feet. Rock size would be from 1 foot to 3 feet at the top surface of the ramp, 0.5 foot to 1 foot in the voids. The weirs would include rock up to 4 feet in diameter. The ramp would start a little below the pool at the base of the dam (el. 445) and terminate at the causeway (el. 454). The ramp would have an average width of 120 feet and would be approximately 180 feet long.

2.6 Alternative E - Add Hydropower

This alternative, suggested by the town of Swanzey, would involve the addition of a small-scale hydroelectric facility to the existing dam as a means to fund the replacement and maintenance of the dam and to possibly raise revenue for the community. Analysis of a hydroelectric development in this location was last completed by Homestead Hydropower, Inc. in 1985 and 1986. This partnership included Mr. Doug Brown, the current owner of the Homestead Dam.

The development plan by Homestead Hydropower in the 1980s envisioned a new electrical powerhouse on the east bank of the river, downstream of the dam. The dam would be reconfigured in a manner so as to divert a portion of the stream flow into a reconstructed forebay, though a new set of turbines in the powerhouse, and thence into a reconstructed tailrace for discharge back to the river approximately 700 feet downstream. This plan was formalized in a 1985 application to the Federal Energy Regulatory Commission (FERC), the agency responsible for regulating such energy projects. **Figure 2.6-1** depicts the plan proposed in this application. However, the FERC apparently never took final action on the application because the plan was determined to be unfeasible (Doug Brown, personal communication).

Note that evaluation of the hydroelectric generating potential of the Homestead Dam is discussed in the last update of the Swanzey Master plan (Swanzey Master Plan Sub-committee, 2003). Specifically, the Master Plan Update references survey results indicating that approximately two-thirds of respondents had positive interest in exploring this possibility. And, the Update references a “feasibility study” by R.A. Greenwood (2000) that calculated that hydropower sold at 5.5 cents/kilowatt-hour would produce net revenues of $40,000 per year.

Hydroelectric plants can provide clean electric power at a relatively low cost. They are somewhat limited in the relative amount of power they can produce when compared to other forms of generation. For example, a single PSNH oil/gas fired power plant in coastal NH produces 409 megawatts of power while PSNH’s nine hydroelectric plants have a combined output of only 69 megawatts. Hydroelectric power therefore comprises only 5 percent of PSNH's total electrical output (Richard Dumore, personal communication, 2004). Some hydroelectric plants have the advantage in that they can store generating capacity. If additional power is needed rapidly on a high-use day, operators can open the dam to create additional power and take advantage of power grid conditions. But, this advantage must be exercised within constraints so as not to adversely impact river flows.

Under the authority of the Federal Power Act, as amended, the FERC has the exclusive authority to license most nonfederal hydropower projects. After a license or exemption from licensing is issued, the FERC monitors the licensee's or exemptee's compliance with the conditions of the license or exemption.
A small-scale hydropower facility such as the one proposed by Homestead Hydropower would potentially qualify for an exemption from the full requirements of federal licensing since it would generate less than 5 megawatts. This exemption applies to projects located at a nonfederal, pre-1977 dam. Additionally, the applicant must show that he or she has all the real property interests necessary to develop and operate the project or an option to obtain the interests.

Even with an exemption, the regulatory process is rigorous and can take some time to conclude. Because of the federal review, the requirements of the National Environmental Policy Act would apply (*i.e.*, an "Environmental Assessment" or an "Environmental Impact Statement" would be required, depending on the scope of the project). In fact, most of the requirements for an exemption are the same as those for a license, with a few specific exceptions. Exemptions from licensing are still subject to mandatory terms and conditions from the Fish and Wildlife Service, the National Marine Fisheries Service, and the state fish and wildlife agency.

So, while hydropower is considered a renewable energy source, and produces no air emissions like other forms of electrical generation, it can have substantial environmental effects on wildlife and water. In recent years, the management of in-stream flows and fishery impacts associated with hydroelectric stations have become increasingly important issues. While certain existing hydropower stations in New England have been re-licensed over the past 15 years, we are not aware of any new facilities that have received FERC licenses or exemptions.

In addition to the construction of a new powerhouse, purchase of a new electrical turbine set, reconstruction and stabilization of the tailrace, and construction of an interconnection to the power grid, this alternative would require replacement of the existing dam and the addition of fish passage facilities. In the current regulatory environment, it is highly unlikely that FERC would approve a new hydropower facility without taking such measures.

2.7 Cost Estimates

The cost of each of the above scenarios was estimated by engineers at Kleinschmidt Associates. Opinions of Probable Cost are based on field survey information, field visits and measurements, the professional construction experience of the Kleinschmidt staff, past costs for similar projects, and cost detail information for each construction task from the 2004 R.S. Means catalog. The R.S. Means Company, Inc. has been publishing construction cost data for over fifty years. Kleinschmidt routinely evaluates bids on behalf of clients for related construction projects, and as a result, is very familiar with the process by which R.S. Means develops its cost data and indices. In fact, Kleinschmidt has routinely used and relied upon the Means cost data to evaluate bids received by clients, and have found such data to be accurate and reliable. Therefore, based on direct experience, the Means cost publications are a reasonable source of data for estimating decommissioning costs.

2.7.1 Construction Cost Estimates

The cost estimates are based on engineering for the Homestead Dam site completed to date. It should be noted that only preliminary conceptual engineering has been completed. Therefore, while the cost estimates are considered accurate and appropriate for a feasibility study of this type, the actual cost associated with any of the alternatives is expected to change as additional engineering is completed on the selected alternative. Nevertheless, the costs estimates are considered a reliable way of assessing the relative economic impact of each option.

Note that the cost estimates provided in **Table 2.7-1** are an initial investment associated with the construction, and do not include life cycle costs associated with operations and maintenance.

Table 2.7-1
Preliminary Construction Cost Estimates, by Option

Alternative	Construction	Contractor Expenses[4]	Engineering[5]	Construction Monitoring[6]	Contingency[7]	Total
Dam Removal[1]	$112,745	$16,822	$11,274	$11,274	$15,000	$167,116
Channel Reshaping[1]	$12,933	$1,940	$2,587	$1,293	$3,000	$21,753
Dam Replacement	$459,581	$43,870	$29,247	$29,247	$84,000	$645,943
Denil Ladder[2]	$169,680	$25,425	$25,425	$16,968	$36,000	$273,552
Bypass Channel[2]	$81,140	$12,171	$16,228	$8,114	$18,000	$135,653
Rock Ramp[3]	$284,140	$42,621	$28,414	$28,414	$58,000	$441,589

Notes:
1. Dam removal costs and channel reshaping must be combined to estimate the total costs associated with a full removal ($188,868).
2. Fish passage cost estimates must be combined with the cost to replace the existing dam ($645,943).
3. The costs for the rock ramp alternative must be combined with the dam removal costs ($167,116).
4. Contractor expenses are assumed to be 15 percent of total construction.
5. Engineering is assumed to be 10 percent of total construction.
6. Construction monitoring is estimated at 10 percent of total construction.
7. Contingency is allowed at approximately 15 percent of total project cost.

Note that several of the cost estimates reported in **Table 2.7-1** must be combined in order to generate the actual costs associated with any particular option. Thus, total construction cost for each alternative is reported below:

2.7.2 Operations and Maintenance Costs

Construction costs can be thought of as one-time expenditures, incurred during the initial stages of a project. However, a true estimate of the cost of a structure must consider its life cycle costs. Life cycle costs take into consideration both construction cost and the cost associated with operation and maintenance (O&M) of the facility.

Table 2.7-2
Total Construction Costs, by Alternative

Alternative	Subtotal	Total
A – No Action		**$0**
B – Full Removal		
Dam Removal	$167,116	
Channel Reshaping	$21,753	
Total Alt B		**$188,869**
C1 – Denil Ladder		
Dam Replacement	$645,943	
Denil Construction	$273,552	
Total Alt C1		**$919,495**
C2 – Bypass Channel		
Dam Replacement	$645,943	
Bypass Construction	$135,653	
Total Alt C2		**$781,596**
D – Rock Ramp		
Dam Removal	$167,116	
Ramp Construction	$441,589	
Total Alt D		**$608,705**

Notes: Component costs are detailed in Table 2.7-1 above and in Appendix D.

Table 2.7-3 reports a summary of expected operations and maintenance associated with the various options that comprise Alternatives A through D. Details of these estimates are provided in **Appendix D**. These costs are estimated over a 30-year time period.

Table 2.7-3
Preliminary Operations and Maintenance Cost Estimates, by Option, 30-year Period

Option	Labor & Materials	Operations	Contractor Expenses	Engineering & Construction Inspection[1]	Contingency[2]	30-year Total
Crib Dam Replair[3]	$185,438	$0	$27,816	$27,816	$36,000	$277,069
Denil Ladder[4]	$5,000	$67,500	$0	$0	$11,000	$83,500
Bypass Channel[5]	$13,613	$67,500	$2,042	$2,722	$13,000	$98,877
Rock Ramp[6]	$25,138	$0	$3,771	$5,028	$5,000	$38,937

Notes:
1. Engineering and Construction Inspection is estimated at either 15 percent or 20 percent, depending on the complexity of the likely repairs.
2. Contingency is allowed at approximately 15 percent of total project cost.
3. Repairs to a replacement crib dam is expected to cost $185,438 for three repair events over a 30-year time period (*i.e.,* every decade).
4. Denil structures are typically low maintenance. This study assumes only minor rebuilds and inspections of the control weir.
5. Assumes one repair to the bypass channel and gates every 15 years.
6. Assumes one repair of the rock ramp every 30 years. No operator costs.

By taking the construction costs together with the operations and maintenance costs, the life cycle costs of each alternative can be derived. Again, it is necessary to add the operations and maintenance of the replacement dam to the costs of O&M for the Denil ladder or the bypass channel to derive the total costs for Alternatives C1 and C2 respectively.

Table 2.7-4
Life Cycle Cost Estimates, by Alternative

Alternative	Construction	O&M	Total (30 years)
A – No Action	$0	$0	$0
B – Full Dam Removal	$188,859	$0	$188,859
C1 – Replacement + Denil Ladder	$919,495	$360,569	$1,280,064
C2 – Replacement + Bypass Channel	$781,596	$375,946	$1,157,542
D – Rock Ramp	$608,705	$316,006	$924,711

Notes: Construction cost estimates are summarized in Table 2.7-1 and detailed in Appendix D. Similarly, operations and maintenance costs are summarized in Table 2.7-3 and detailed in Appendix D.

From **Table 2.7-4**, it is clear that full dam removal is by far the least expensive option, aside from the "No Action" alternative (which is not acceptable, but included for comparison). Dam removal is expected to cost less than $200,000 over the next 30 years, whereas all other alternatives would range from about $0.9 million to almost $1.3 million. The majority of the costs for all alternatives occur during the construction phase, although O&M costs comprise approximately 1/3 of the cost of Alternatives C and D.

2.8 Hydropower Economics

The cost implications of Alternative E (hydropower) differ from other alternatives, since this option is capable of producing income to offset capital and operating expenses. Rather than address only the likely construction costs associated with this alternative, a slightly different approach is warranted. While a full evaluation of potential hydropower costs, benefits, and impacts is beyond the scope of this current study, we have reviewed information from the 1985 Homestead Hydropower, Inc., including the original application and the numerous regulatory responses, as well as the analysis presented by R.A. Greenwood in his June 28, 2000 letter to the Swanzey Selectmen. We also draw on experience with previous analyses of hydropower at other sites.

Below is a brief run down of the finances relative to the potential of converting the Homestead Dam to a hydroelectric generating facility. We have compared our projected costs with those outlined in R.A. Greenwood's letter where appropriate. As in that report, a 30 year payback scenario is envisioned.

Rebuilding of exiting crib dam

As discussed in Chapter 1, an engineering evaluation by Tom Kahl, PE of Kleinschmidt Associates determined that the existing crib dam is in such disrepair it will need complete replacement. The estimated cost of that replacement totals includes $167,116 to remove the existing dam and $645,943 to construct a new dam in its place. Note that, because the existing dam clearly does not meet FERC guidelines for safety, partial replacement of the scope contemplated by the RA Greenwood report is unlikely to satisfy regulatory requirements if an application for exemption were to be filed. Thus, the total cost to dam replacement is $813,059 or $27,102 per year as compared to the R.A. Greenwood cost of $35,000 or $1,167 per year.

Fish Passage

We assume that any exemption issued by the FERC will require the installation of fish passage. For this exercise, we have used the Denil style fish passage since it will best accommodate the layout constraints due to the location of the powerhouse. That cost is $273,552 or $9,118 per year. There is no fish passage cost item in the R.A Greenwood report.

Powerhouse Construction

Construction of a new powerhouse is likely to be $3,000 per installed kilowatt (assume 655 kw) or $1,965,000 or $65,500 per year. The R.A. Greenwood report used a cost of $1,385,000 or $46,167 per year.

Operation & Maintenance

Maintenance costs of the crib dam is based on conversations with the dam's present owner, Kleinschmidt's professional experience, and recommendations from Hydroelectric Handbook by Creager and Justin (1950). In this scenario, per these resources, the dam will require complete replacement in 25 years and regular maintenance several times within that 25 year period. The annual cost for crib dam maintenance is $21,530 per year. R.A. Greenwood assumed $2,000 per year. Operation of the powerhouse and fish passage would be $42,000 year with benefits. The R.A. Greenwood report used a cost of $32,000 per year.

Revenue Generation

R.A. Greenwood uses a scenario where the revenue for power sales would be approximately 5.5 cents per kilowatt hour. While this price may occur at some point in the future, a more conservative analysis would be based on the present rate of 4.5 cents per kilowatt hour with 2300 mwh produced per year. Thus, our estimate is that a hydroelectric facility would produce approximately $103,500 per year in revenue.

Excluded Costs

Note that we have not included the cost of maintenance to the powerplant or licensing/permitting expenses. In addition to the FERC process described above, the project would also need approval through the US Army Corps of Engineers and the NH Wetlands Bureau.

A summary of expected economics of a hydropower facility at this location is contained in **Table 2.8-1.**

Table 2.8-1.
Hydroelectric Costs and Revenues, Annualized at 30 years

Cost/Revenue	**Total Ann. Cost ($)/yr** *or* **Surplus $/yr**
Rebuild Dam	($27,102)
Fish passage	($9,118)
Powerhouse construction	($65,500)
Dam maintenance	($21,531)
Operations	($42,000)
Sale of electricity generated (4.5 cents/kwh)	$103,500
Net annual economic loss	**($61,751)**

The above analysis suggests that installation of hydroelectric facilities to the Homestead Dam would not likely result in an economic benefit to the community.

Additionally, it may be difficult to obtain a FERC license/exemption for this site due to the project's minimal generating capacity and the fact that it has low and no flow situations (and therefore generating capacity) when needed most, in the summer months. Per historical stream flow data by USGS there will be 3 to 4 months each year when there is no power generation due to the minimum flow requirements of the turbine and water requirements of the fish bypass system. Though there have been upgrades to existing hydro generating plants, to our knowledge there have been no new facilities constructed in New England in the last 10 to 15 years.

Note that this conclusion agrees with the ultimate outcome of the Homestead Hydropower analysis, which eventually found that the amount of head and flow, and therefore potential power generation, did not justify the investment (Doug Brown, personal communication, 2004).

3

Impacts Analysis

3.1 Introduction

A variety of alternatives have been developed to address the goals of this project. This chapter includes information relative to the evaluation of each of the alternatives discussed in Chapter 2, including a discussion of existing environmental conditions, methods of analysis, and major conclusions.

The alternatives analysis includes consideration of environmental and cultural resources as well as analysis of the engineering constraints and projected operations associated with each alternative. Although this Feasibility Study provides a full analysis of these constraints, it is important to note that each alternative has been designed only to a conceptual level. Quantitative analysis is presented where possible, while some analyses are of a more qualitative nature.

The main difference among alternatives relates to their potential effects on the size and depth of the dam impoundment. In examining the full range of alternatives, it should be noted that they can be classified in one of two ways: 1) either the alternative would lead to elimination of the impoundment, as is the case for Alternatives A and B; or 2) the alternative maintains the impoundment, as is the case for Alternatives C, D, and E. Thus, much of the discussion below is presented with this major distinction among the alternatives in mind. We refer to these two cases as the “dam in” and “dam out” scenarios.

The discussion below begins with a description of the fluvial geomorphic setting of the river. The results of hydrological and hydraulic analysis of the river are also presented. Once these analyses are understood, their results can be extrapolated to determine effects on environmental and cultural resources.

3.2 River Geomorphology, Hydraulics and Sediments

The drainage area above the Homestead Dam to the crest of the spillway is 316 square miles. Information on the height of the dam is inconsistent among different sources. Most sources cite a dam crest elevation of 456.2 feet, and provide a height of between 10 feet and 14 feet. Survey performed in the summer of 2004 found that the actual height of the dam crest (spillway) varies along its length due to settlement of the structure. In most places, the crest is approximately elevation 455 (NGVD 1929), with spot grades along the crest varying from elev. 454.65 to elev. 455.50. The crest stands approximately 12 feet above the streambed immediately downstream of the toe of the dam.

According to information contained in the DES dam file and other sources, the dam affects flow for approximately 3.4 river miles upstream from the dam. The surface area of the impoundment at the present dam crest elevation is ±45 acres. This creates a long, narrow impoundment with storage of 158 acre/feet and an average depth of 3.5 feet.

Observation of USGS data indicates that the peak annual flow period over the last thirty years generally has occurred in late March and early April. The lowest flow-period has historically been recorded in very late August and early September. Flows recorded at the West Swanzey gauging station in the last five years have ranged from approximately 3,350 cubic feet per second (cfs) to under 10 cfs.

The fifty-year flood event level (Q_{50}) has been determined to be approximately 6,190 cfs (see HEC-RAS model discussion below). Peak flows were estimated at 13,400 cfs in 1936, when the water depth was 8 feet over the crest of the spillway.

3.2.1 Fluvial Geomorphic Setting

The geomorphology of the impounded reach of the Ashuelot was inspected by boat in August 2004. Additionally, the discussion below is based on consultation of current aerial photographs, a historical topographic map surveyed in 1895, and historical ground photographs. The site visit included reconnaissance of an area six miles upstream and three miles downstream of the dam with depth soundings (using a Hondex Digital Depth Sounder) taken periodically upstream while floating the river by boat.

Figure 3.2-1 depicts the major geomorphological features of the Asheulot River in the project reach. In the vicinity of the dam, the river flows across a floodplain bordered on both sides by river terraces formed, most likely, during deglaciation of the region approximately 12,000 years ago. The river impinges against these terraces in several places but flood flows maintain access to a wide floodplain on the opposite bank between the city of Keene and the confluence with the South Branch of the Ashuelot River. The floodplain is much narrower downstream of the South Branch confluence. The floodplain is narrowest at the dam itself, which probably explains why the dam was originally constructed at this spot. Where the floodplain is wide, large floods will have largely the same effect on channel morphology as more frequent, nearly annual, bankfull events because the excess flow of the larger floods spreads across the floodplain where its force is greatly diminished. As the floodplain becomes narrower and flows more greatly constricted, the potential for greater scour in the channel will increase with increasing discharge. When the flow across a wide floodplain encounters a narrower portion of the floodplain downstream, as is the case between the South Branch confluence and the dam, a backwater effect will occur at the upstream end of the constricted area. Under natural conditions, this increased channel scouring and backwater effect is probably significant in only very large flows since the floodplain is still more than two times the width of the channel at its narrowest spot at the dam. However, the mill building associated with the dam occupies the entire floodplain on the right bank of the river (looking downstream) with the upstream end of the building located where the Thompson Covered Bridge crosses the Ashuelot River. On the left bank, the river flows against a terrace approximately 5 feet higher than the floodplain on the right bank. Consequently, all flow on the right bank floodplain is forced under the Thompson Covered Bridge, increasing the potential for scour at the bridge and creating a backwater effect upstream. In extreme cases, the backwater effect would eventually cause the terrace on the left bank to be overtopped.

The dam is approximately 10-12 feet higher than the bed of the channel immediately downstream, creating a slight impoundment at low flow that extends some distance upstream. The impounded waters are all maintained within the channel banks so the original channel form is still visible.

Geomorphic observations of the impounded reach provide four lines of evidence that suggest that the dam does not significantly alter water or sediment flow during bankfull flow conditions and higher, except perhaps in the immediate vicinity of the dam:

- First, water depths in the impounded area are generally much greater within the meander bends than in straight reaches. This is due to the creation of secondary flow circulation cells in meanders with a downward projection along the outside bend of meanders.

- Second, bank erosion is most prevalent on the outside bends of meanders, as in unimpounded rivers elsewhere, because these downwardly projected circulation cells serve to undermine the bank and cause its eventual collapse.

- Third, extremely deep pools over 20 feet deep were encountered where the river impinges on the terraces fringing the floodplain. One of these deep pools is only 1,800 feet upstream of the dam. While this represents the depth below an impounded water surface, these pools are also 15 feet deeper than adjacent areas on the channel bed. The greater resistance encountered by the river as it flows against a high bank results in the river's energy being expended on the more erodible bed, creating the deep pools observed. These pools would tend to fill with sediment if significantly affected by an impoundment.

- Finally, comparison of the 1895 topographic map and 1998 digital orthophotograph reveals some growth in the meanders 1.0 mile upstream of the dam. This indicates that the impounded area has been more or less free flowing, at high flows at least, for some time.

Additional understanding of the river can be gained through computer modeling of the river, as discussed below.

3.2.2 Development of a HEC-RAS Hydraulic Model

In order to evaluate the changes in water depth, width and velocity if the dam were to be removed, a backwater model of the Ashuelot River upstream and downstream of the Homestead Dam was prepared using the US Army Corps of Engineers' HEC-RAS program, version 3.1.2. HEC-RAS is designed to perform flow calculations in natural and man made channels, as well as to perform unsteady flow routing and elementary sediment transport computations. The model can simulate depths and velocities for a single reach, a branched system, or a full network of channels, and can simulate sub-critical, super-critical, and mixed flow regimes.

The HEC-RAS model for this project included 69 cross sections that extended from approximately three miles downstream of the dam (at the Swanzey-Winchester town line) to a distance of nearly seven miles upstream of the dam (approximately one mile above the Keene-Swanzey town line). The locations of a selected number of these cross sections are shown at key locations in the study area on **Figure 3.2-2**. The model included the Homestead Dam and several bridges crossing the Ashuelot River, including the Thompson Covered Bridge.

The model can be used to help answer the following questions:

- What are the water surface elevations and velocities in this reach of the Ashuelot River under different flow events for "dam in" and "dam out" conditions?
- Could water velocities under dam out conditions scour existing infrastructure such as the Thompson or Cresson Covered Bridges?
- If the dam is removed, will water levels drop to an extent that recreational or natural resources might be affected?
- Will wells adjacent to the river be affected?
- Will water depths and velocities be sufficient for fish to pass through the project area if the dam is removed?
- Will changes in water velocities cause sediment to migrate downstream?

Each one of these questions is considered in this Feasibility Study. First, however, it is important to understand how the model was built as well as its basic findings.

3.2.2.1 Hydraulic Model Input

To build the HEC-RAS model, channel and overbank geometry, as well as hydraulic roughness ("Mannings *n*") was derived from three primary sources, as follows:

- Cross sections downstream of the dam were derived from the HEC-RAS modeling forwarded to VHB & Kleinschmidt by DES. The modeling was prepared by CLD Consulting Engineers in 2002 for their study of scour at the Thompson Covered Bridge. The cross sections downstream of the dam were originally used in the Federal Emergency Management Agency (FEMA) flood insurance studies prepared for the Town of Swanzey in 1981.

- Cross sections immediately downstream and upstream of the dam, including the Thompson Covered Bridge, were derived from the VHB topographic and bathymetric surveying conducted in 2004. There were 12 cross sections extending approximately 400 feet downstream of the dam, and 14 cross sections upstream of the dam (including the cofferdam/access road) extending to just above the Thompson Covered Bridge. Four additional surveyed cross sections upstream and downstream of the dam were also used in the HEC-RAS modeling. They included a cross section 620 feet downstream of the dam, at a riffle, as well as cross sections 2,500 feet, 5,600 feet and 8,100 feet upstream of the dam.

- Cross sections upstream of the dam were derived from HEC-RAS modeling prepared by Delta Environmental Services for a flood insurance study for the City of Keene in 1997. While some of the cross sections were the same as used in the City of Keene flood insurance study originally prepared in 1983, the 1997 HEC-RAS data forwarded to VHB and Kleinschmidt by the DES contained new and revised cross sections reflecting geometric changes (such as meander and oxbow cutoffs) that have occurred since 1983. The use of this data allowed the HEC-RAS model to be extended to nearly 1 mile upstream of the Swanzey-Keene town line and thereby capture the upstream extent of backwater influence from the Homestead Dam.

Cross section data was referenced to a common USGS datum for use in the HEC-RAS model.

The HEC-RAS model was run with and without the Homestead Dam for several flow scenarios. A 90 percent exceedance low flow, 60 cubic feet per second (cfs), as derived from the USGS streamgage records, was the lowest flow modeled. Other modeled flows included the annual mean flow (520 cfs), as well as the May, June and July mean flows (750, 400 and 180 cfs, respectively), which capture the upstream migration period in the spring for American shad, alewife and blueback herring. "Bankfull flow" was approximated as having a recurrence interval of 1.5 years, which is typical for alluvial rivers. As approximated from streamgage records, the bankfull flow was 2,600 cfs. Flood flows were derived from streamgage data and the flood insurance studies, and included the 2-year flood (2,940 cfs), 10-year flood (4,630 cfs), 50-year flood (6,190 cfs) and 100-year flood (6,840 cfs).

Starting water levels at the downstream end of the model were either derived from flood insurance studies (for the 10-year, 50-year and 100-year floods) or were assumed to be critical depth. Given that the downstream end of the model was three miles below the Homestead Dam, water levels upstream of the dam were not sensitive to the starting water surface elevations that far downstream. For the condition with the dam in place, water levels at the dam were calibrated to the USGS rating curve for the streamgage just upstream of the dam. The rating curve reflects current conditions, such as the continuing deterioration of the Homestead Dam and flow through the dam. For the dam removed condition, it was assumed that the dam and cofferdam (access road) upstream of the dam would be removed, with the channel in the vicinity of the dam reshaped as a boulder riffle to approximate historic conditions and to facilitate fish passage.

3.2.2.2 HEC-RAS Results

Several hydraulic parameters were calculated by the HEC-RAS model at each cross section for the two conditions (dam in and dam out) and various flows. The hydraulic parameters included water level, channel depth, channel and overbank velocities, channel and overbank shear stresses, wetted top width, cross sectional area and slope of the energy grade line. Calculations for the reach upstream of the dam (41 cross sections) included total surface area and volume. All of these parameters may be important for understanding the potential effects of dam removal. Velocity, for example, may be important for understanding streambank erosion and sediment transport for different dam conditions and flows. Changes in total reach surface area and volume may similarly be important for understanding impacts to wetlands and anadromous fish spawning habitat.

Table 3.2-1 summarizes the predicted changes in the impoundment volume and area under dam repair and dam removal scenarios.

For the most part, the results are not unexpected. The degree to which the Homestead Dam acts as a hydraulic control diminishes significantly as the flow increases. That is, the most abrupt changes in volume and surface area if the dam were to be removed would be for low flows. For higher flows beginning at about 2,000 cfs (a spring flow in West Swanzey) the changes after dam removal would be relatively unsubstantial throughout the reach.

Table 3.2-1
Mean Flows, Impoundment Volume & Area, Under Existing and Dam Removal Scenarios

		Impoundment Volume (acre-feet)				Impoundment Area (acres)			
Flow Condition	**Q (cfs)[1]**	**Existing**	**No Dam**	**Change**	**% Decrease**	**Existing**	**No Dam**	**Change**	**% Decrease**
90% Exceedance	60	371	146	225	61%	86	71	16	18%
July Mean	180	424	227	197	46%	91	81	10	11%
June Mean	400	504	330	174	34%	96	88	8	9%
Annual Mean	520	543	378	165	30%	99	91	8	8%
May Mean	750	616	462	153	25%	105	97	7	7%
Bankfull	2,600	1,472	1,271	202	14%	481	430	51	11%
2-Year Flood	2,940	1,697	1,475	221	13%	515	477	38	7%
10-Year Flood	4,630	2,814	2,556	258	9%	656	609	48	7%
50-Year Flood	6,190	3,863	3,605	258	7%	739	720	18	2%
100-Year Flood	6,840	4,302	4,059	243	6%	771	754	18	2%

Notes:
1. "Q" denotes the flow in the river under various categories in cubic feet per second. These flows were determined primarily from stream gauge data maintained by the USGS.

For the lowest flows, say the typical July flow of about 180 cfs, the volume of water stored by the dam would decrease by almost half. This might lead one to envision a dramatically smaller river. But, because of the flat slope of the valley and the cross-section of the channel throughout most of this reach, the surface area of the impoundment would not decrease nearly as significantly. From **Table 3.2-1**, it can be seen that the surface area of the Homestead Dam impoundment will decrease by only 11 percent under typical July flows with the dam out, despite the fact that the volume of water held by the impoundment drops by about half. **Table 3.2-2** shows additional summary data that highlights that the width of the stream does not decrease substantially under these low flows. For example, the July mean stream width will decrease from about 103 feet wide to about 84 feet wide, or 11 percent. The depth of the stream, however, will decrease rather substantially, from an average of about 4.65 feet in its impounded status to about 2.81 feet if the dam is removed. This figure must be interpreted very carefully, however, since it is averaged over the length of the impoundment.

Thus the volume and surface area upstream of the dam changes relatively little during spring flows and greater. Even though the mean depth may decrease appreciably, the presence of many deep pools and deadwaters - and the overall low gradient nature of the river - means that the area and the width of the stream do not change a lot during spring flows.

These results indicate that the Ashuelot is retaining riverine characteristics - submerged riffles and pools, channels deeper on the outside of bends, *etc*. - even though it is impounded. The riverine dynamics are perhaps dampened by the

Table 3.2-2
Mean Channel Widths and Flow Depths, Under Existing and Dam Removal Scenarios

		Mean Channel Width (feet)				Mean Stream Depth (feet)			
Flow Condition	Q (cfs)[1]	Existing	No Dam	Change	% Decrease	Existing	No Dam	Change	% Decrease
90% Exceedance	60	103	84	19	18%	4.29	2.06	2.24	52%
July Mean	180	108	96	12	11%	4.65	2.81	1.84	40%
June Mean	400	114	104	10	9%	5.25	3.77	1.48	28%
Annual Mean	520	118	108	9	8%	5.49	4.16	1.33	24%
May Mean	750	124	116	9	7%	5.88	4.74	1.13	19%
Bankfull	2,600	572	512	60	11%	3.06	2.95	0.11	4%
2-Year Flood	2,940	612	567	45	7%	3.30	3.09	0.20	6%
10-Year Flood	4,630	780	724	56	7%	4.29	4.20	0.09	2%
50-Year Flood	6,190	878	856	22	2%	5.23	5.00	0.23	4%
100-Year Flood	6,840	917	896	21	2%	5.58	5.38	0.19	3%

Notes:
1. "Q" denotes the flow in the river under various categories in cubic feet per second. These flows were determined primarily from stream gauge data maintained by the USGS.

impoundment, but by no means is this river reach a deepwater lacustrine system that has filled in with muck and other soft sediments.

That is not to say that the hydraulic changes are insignificant. Velocities do go up appreciably is many areas, and of course the bridges are a concern. Interestingly, the data in **Appendix E** show that velocities will go up appreciably in certain river reaches as some of the historic riffles emerge and the flow tries approaches a supercritical state.

3.2.3 Tractive Force Analysis

Rivers move sediment along with water. Sediment transport is a naturally occurring, continuous process in all streams. Typically, streams are in dynamic equilibrium between sediment deposition and scour, usually resulting in a stable channel configuration. Local changes in this equilibrium can result from, among other things, high flow events, erosion from adjacent upland sources, or changes to the hydraulic characteristics of a river reach due to new or changed infrastructure (*e.g.*, a bridge or culvert). Just as rivers move sediment in addition to water, dams impound sediment just as the impounded water. Thus, it can be assumed that some amount of sediment migration would accompany dam removal.

Tractive force analysis is a methodology to assess potential changes in the way that the river would transport sediment under the dam out condition. Such an analysis was prepared using HEC-RAS model results for the bankfull flow, which was approximated as a high flow (2,600 cfs) with a 1.5-year recurrence interval. Bankfull flow, which would occur almost annually, is sometimes referred to as the "channel forming" or "channel maintenance" flow because it is highly influential in determining channel width, shape, planform (*e.g*, bends and meanders) and the gradation of streambed substrates. It is also the flow that has a high enough velocity to initiate particle motion but is shallow enough to allow turbulent flows to interact with the channel bed and is therefore representative of the flow most significant in moving sediment.

Tractive force, related to shear stress, is proportional to the slope of the water surface and the depth of the flow. Using the water surface slope and maximum channel depth from the HEC-RAS model results, it is possible to calculate the tractive force at each cross section. For non-cohesive bed materials greater than one centimeter in diameter, the tractive force (in kilograms per square meter) is approximately equal to the incipient diameter of the streambed particles (in centimeters), a guideline sometimes referred to as Lane's relationship. The incipient diameter is the diameter at which individual particles subjected to a shear stress begin to move. While sediment transport is a very complex phenomenon, changes in tractive force from one cross section to another, or from one condition to another (*e.g.*, dam repair *vs.* dam removed), may predict changes in sediment transport and channel maintaining processes.

For a given tractive force, the corresponding incipient diameter of the substrate can be classified using any of several soil classification systems. For this analysis, the Unified Soil Classification System (USCS) was used. The soil gradations for the USCS are as follows:

Table 3.2-3
Universal Soil Classification System

Sediment Type	Diameter (mm)
Fines (Silt, Clay)	< 0.075
Fine Sand	0.075 - 0.425
Medium Sand	0.425 - 2.00
Coarse Sand	2.00 - 4.75
Fine Gravel	4.75 – 19
Coarse Gravel	19 – 75
Cobbles	75 - 300
Boulders	> 300

By comparing the incipient diameters between the existing (dam in) and proposed (dam out) conditions, a comparison can also be made between the soil classification at any given cross section. Large increases in the incipient diameter may be predictive of changes in substrate size and channel geometry. For example, if the incipient diameter at a cross section goes from fine gravel to cobbles after dam removal, there may be significant scouring of bed material--and perhaps streambank erosion--at this section. This may also indicate a morphological change at this section from a shallow pool or run to a riffle.

Since Lane's relationship is most accurate for incipient diameters greater than ten millimeters (fine gravel), changes in soil classification between sand gradations are probably not significant predictors of channel changes. Increases in tractive force may indicate a coarser sand that is able to be transported after dam removal, and perhaps increased bank scour. More significant, however, are large jumps in soil classification from gravels to cobbles or boulders, which are indicative of large increases in bed mobility.

3.2.3.1 Tractive Force Results

For the bankfull flow, the HEC-RAS model indicates that the removal of the Homestead Dam would increase the soil classifications of the incipient diameters at several cross sections upstream of the dam. Upstream of the Thompson Covered Bridge (cross sections 41 through 69), most of the changes are insignificant, with the incipient diameters after dam removal remaining fine gravel or smaller. The incipient diameter would increase to coarse gravel at cross sections 42, 46 and 56. At cross section 41, just upstream of the Thompson Covered Bridge, the incipient diameter would increase from cobbles to boulders after dam removal.

The most significant increases in incipient diameter occur between the Homestead Dam and the Thompson covered bridge (cross sections 29 through 40). Just downstream of the bridge (cross section 40), the incipient diameter increases from coarse gravel to boulders. Within this 350 foot reach, the incipient diameters after dam removal would be predominantly cobbles and boulders, as would be expected for a riffle.

Downstream of the Homestead Dam, the incipient diameters appear to be mostly sands. Based on field observations, the river downstream of the dam does have gravel, cobbles and boulders, with the riffles highly embedded with sand. This seems to indicate that the larger substrates (coarse gravels through boulders) downstream of the dam are acting as a relatively stable “pavement,” and are not highly mobile.

3.2.4 Potential Fluvial Response to Dam Removal

Sediment transport is a complex phenomenon. For any given reach in the Ashuelot River, the streambed is comprised of a mix of substrates, including sand, gravels, cobbles and boulders. Sediment transport and channel changes are not only related to the shear stress (tractive force) for any given flow, but also to the sediment volumes input from streambanks and mobile beds. Ice dynamics can also be a factor, with moving blocks of ice, scouring streambanks or anchor ice picking up and moving cobbles and some boulders. However, the tractive force analysis is useful for highlighting reaches where substrates may become a lot more mobile after dam removal. As noted above, the reach between the Thompson Covered Bridge and the Homestead Dam may be particularly subject to scour after dam removal, as the incipient diameter increases from gravel to cobbles and boulders.

Table 3.2-4 Tractive Force Analysis Summary

	Dam In			Dam Out			Incipient
Landmark/River Station	Tractive Force (kg/m^2)	Incipient Diameter (mm)	USCS Soil Classification	Tractive Force (kg/m^2)	Incipient Diameter (mm)	USCS Soil Classification	Diameter Increase (in)
Cresson Bridge	0.00	0.00	fines	0.00	0.00	fines	0.00
Indian Fishing Weir (approx)	0.64	6.44	fine gravel	0.82	8.19	fine gravel	0.07
Meander Bend (~1 mile upstream)	0.37	3.68	coarse sand	0.50	4.99	fine gravel	0.05
Spring St. terminus (~½ mile upstream)	0.09	0.87	med sand	0.21	2.12	Coarse sand	0.05
Thompson Covered Bridge	2.94	29.44	coarse grav	36.53	365.34	boulders	13.22
Between Dam & Bridge (150 feet upstream)	0.00	0.00	fines	2.41	24.14	Coarse grav	0.95
Homestead Dam	-	-	-	4.37	0.90	med sand	1.79
D. Thompson Bridge (~¼ mile downstream)	0.68	0.14	fine sand	0.68	0.14	fine sand	0.00
Westport (~3 miles downstream)	1.12	0.23	fine sand	1.12	0.23	fine sand	0.00

The increase in tractive force in this reach after dam removal has several implications. The first is that an increase in the incipient diameter just downstream of the Thompson Covered Bridge may lead to increased scour in this area, depending on the existing bed material. In fact, this scour may already be increasing as the timber crib dam and access road (cofferdam) continue to erode and lower water levels upstream of the dam. The topographic survey and HEC-RAS model indicate that this reach is one of the steepest along the Ashuelot River, and may have been a prior to dam construction. This conclusion agrees with some historical accounts that describe a cascade or falls in the West Swanzey area.

After dam removal, the finer materials (especially sands) in this reach may be quickly scoured down to coarser material, such as large gravels, cobbles and boulders. This bed material would probably first fill the scoured area just downstream of the dam, but may eventually migrate downstream. Given that the movement of a large amount of bed material in this reach may be undesirable, it is recommended that stream restoration techniques be considered in the event of dam removal, including the design and restoration of a sustainable thalweg, bed slope, channel slope and bed gradation in this reach.

The results of the tractive force analysis agree with predictions based on geomorphic observations and past experience with dam removal. For example, a detailed one-foot contour topographic map of the dam site, with numerous spot elevations below the water surface of the river, indicate that the channel thalweg (*i.e.,* deepest part of the channel) is three to four feet higher immediately upstream of the dam compared to just downstream. Consistent with this, a photograph of the dam during an 1880 flood shows what appears to be an approximately three foot elevation difference in the water surface upstream and downstream of the bridge.

This grade change at the dam is important to note because it indicates that dam removal could initiate a headcut[1] that would lower the stream bed three feet immediately upstream of the dam, including the area around the bridge, if measures to prevent such an occurrence are not in place immediately following removal. Careful engineering of a new stable channel would therefore be necessary in this area. Otherwise, the headcut might continue upstream until it diminished to zero near the upstream end of the impoundment or where changes in tractive forces reached insignificant levels.

A headcut could, in turn, cause the river banks to be undercut, destabilizing the banks. This would be most noticeable immediately upstream of the dam along unvegetated banks where the lack of tree roots limits bank strength necessary to withstand minor undercutting. Further upstream, the amount of bed lowering is unlikely to be great enough to initiate large bank instabilities. Minor incision of the channel bed would likely cause the current channel position to become more stable and channel avulsions and rapid channel migration less likely. It is important to note that increases in sediment delivery to the channel generally drive rapid channel changes, a condition that would be unaltered by dam removal upstream of the dam.

Lowering of the channel bed would not progress as described above if, during the incision, the channel encounters a resistant substrate such as bedrock or large boulders that the river is not competent enough to transport. While the completed geotechnical investigations do not indicate the presence of bedrock at or near the surface in the vicinity of the dam, glacial till with cobbles to large boulders is likely present near the surface. It is not clear from the geotechnical borings whether a sufficient number of boulders occur in the till to armor the channel bed and arrest erosion before the headcut migrates substantially. For this reason, any alternative that involves removal of the dam should plan to place stream bed armoring between the dam and the bridge to prevent channel headcutting. This artificial armoring would be constructed at a slope that would permit fish passage, as depicted in **Figure 2.2-1.**

With removal of the dam, erosion of the channel bed upstream could increase sediment delivery to areas downstream of the dam. This excess sediment will accumulate where velocities and therefore sediment transport is limited, such as in areas experiencing backwater effects (*e.g.,* upstream of bridges, valley constrictions, or tight bends), zones of expanding flow, or existing locations of high sediment input (*e.g.*, at the confluence of tributaries). A mid-channel bar is currently present just upstream of the Denman Thompson Bridge located approximately 1,300 feet downstream of the dam, suggesting that a backwater effect develops at high flows behind the bridge. This represents a likely location for additional sediment accumulation if the dam is removed. Other places include the confluence of Indian Brook. The confluence is the only area where the Ashuelot River has experienced

1 A "headcut" is an erosional feature caused by scouring of the streambed that migrates upstream until the system reaches equilibrium or the headcut encounters a non-erodible surface.

significant channel migration downstream of the dam since 1895, indicating sediment inputs may already be high. With the bar growth accompanying the accumulation of sediment in these depositional areas, flow could be diverted into the channel banks, which may cause erosion. Once the river has achieved a new equilibrium bed elevation upstream, the sediment delivery downstream will return to current levels and the adverse impacts resulting from the sedimentation diminish.

3.2.5 Potential Sediment Contamination

The Ashuelot River watershed has a long history of industrial activity, which in some cases has caused environmental contamination. Even though proper measures can limit the rate at which sediment is carried downstream to normal rates, the release of sediments contaminated by pollutants is a concern to the human and aquatic environment. Before dam removal occurs, it is common to conduct sediment testing for pollutants.

In 1999 and 2000, the US Fish and Wildlife Service conducted sediment testing in the impoundment immediately upstream of the Homestead Dam to search for potential contaminants. The USFWS collected the samples using a stainless steel Ponar Dredge. This dredge is intended for harder sediments than the Eckman Dredge typically used in lakes and ponds. It takes a single, large conglomerate sample of the sediments that it penetrated upon deployment. The sampling team did have a KB corer on hand, but opted not to use this equipment due to the sediment composition encountered (largely unconsolidated material and sand) and the fact that stratified samples were not required to meet the objectives of the screening-level sampling.

Once collected, the samples were analyzed for the following contaminants of concern:

- Metals (*e.g.*, Arsenic, Barium, Cadmium, Chromium, Copper, Lead, Mercury, Nickel, and Zinc);
- Organochlorides, including DDT and its metabolites, other pesticides, and PCBs; and
- Polynuclear Aromatic Hydrocarbons (PAHs), including petroleum products, coal tar derivatives, and other anthropogenic sources.

Samples were delivered to laboratories at the USFWS Pawtuxent Analytical Control Facility and the Geochemical and Environmental Research Group of Texas A&M University. Results for metals were compared to criteria in Long, *et al*. (1995), Ingersoll, *et al*. (1996) and the Ontario Sediment Quality Guidelines. This review indicated that none of the contaminants were found at elevated levels that could pose a risk. See John Warner's (USFWS) memorandum to Jim McCartney dated March 15, 1999 and Drew Major's (USFWS) memorandum to Doug Brown dated May 1, 2000 in **Appendix F.** Based on these findings, additional sediment testing was determined to be unnecessary by the NH Department of Environmental Services (Stephanie Lindloff, NHDES, personal communication, 2004).

3.2.6 River Ice

As discussed above, rivers carry water and sediment. In cold climates, ice can also be an important factor. An ice jam is a stationary accumulation of fragmented ice that restricts river flow. Ice jams are a common, natural process in northeastern rivers, and usually do not cause any problems except when combined with floodplain encroachment by buildings, roads, or bridges. Research by the US Army Corps of Engineers Cold Regions Research & Engineering Laboratory (CRREL) in Hanover, NH has shown that dam removals in the Northeast can affect ice dynamics in certain rivers and therefore the formation of ice jams.

A review of databases maintained by the Army Corp of Engineers Colds Regions Research and Engineering Laboratory (CRREL) revealed that fifteen ice jams have occurred on the Ashuelot since 1935. The majority of these ice jams occurred in Gilsum, approximately 20 river miles upstream (n=10). Two ice jams occurred in Hinsdale, about 18 river miles downstream. The Homestead Dam does not influence ice dynamics at these distances.

However, three ice jams have been reported at the Homestead Dam itself since 1995. Given that all three database reports are from USGS gauge data, which has only been in operation since April 1994, it is possible that other jams have occurred but gone unreported. It is significant to note that local emergency officials do not recall these ice jams causing any hazardous flooding. Thus, while the gauge data may show ice jam formation and may have raised water surface elevations within the river banks, the ice jams have not created hazardous flooding conditions.

Based on ice observations made during the spring of 2004 and winter 2004-2005, it is clear that the Homestead Dam does influence ice dynamics. Specifically, the dam slows the river velocities enough, particularly in the immediate vicinity of the dam, so that sheet ice develops along the impoundment. The contribution of frazil ice (*i.e.*, a form of unconsolidated river ice formed in open, turbulent waters) does not seem to be as significant as with higher gradient streams. Although little information is available on the three previous ice jam occurrences, it seems most likely that these ice jams formed during mid-winter or spring thaws when thermal expansion caused break up of upstream sheet ice concurrent with high flows caused by melted snow and precipitation. The dam prevents the ice floes from continued downstream movement, instead forming jams.

Removal of the dam will remove the barrier to ice flow in this location. Further, changes in the depth and velocity of winter flows may prevent the formation of sheet ice in this reach, which may attenuate the ice dynamics. However, it is unknown if removal of the dam could increase velocities enough to promote frazil ice development, which would produce ice that may have downstream effects. Thus, the potential for ice jam formation at downstream locations (*e.g.*, the Denman Thompson Bridge) is a potential concern.

3.3 Groundwater Resources & Wells

The potential effect to local wells is a substantial concern to Swanzey citizens who live along the river, particularly since these residents depend on private wells for their drinking water. The concern is that a decrease in the impoundment will lead to a drawdown in the local water table which would endanger the yield of residential wells. A hydrogeologic evaluation of the potential effect on private water supply wells is presented in this section to determine the possible extent of these effects.

For purposes of the description and analysis that follows, the drainage divide of the sub-basin where the dam is located defines the upstream extent of the hydrogeologic study area, while the dam defines the downstream extent of the hydrogeologic study area. The width of the hydrogeologic study area is the areal extent of the mapped sand/gravel aquifer deposits as depicted in **Figure 3.3-1**.

3.3.1 Hydrogeologic Existing Conditions

3.3.1.1 Geologic Setting

The three types of aquifer materials which can supply water for water supply wells include stratified sand and gravel deposits, till and bedrock. Sand/gravel deposits have a much higher hydraulic conductivity than till or bedrock, and therefore usually yield higher quantities of water. Till is less productive than the sand/gravel deposits and can only supply minor amounts of water for individual private household wells. Bedrock can supply variable amounts of water.

The underlying bedrock in Swanzey is composed of bedrock domes with gneiss or granite cores. Bedrock is covered by low permeability till in most locations and the water bearing fractures are deep in the study area. A review of well logs in the vicinity of the dam, available from DES, shows that the bedrock is approximately 80 to 100 feet deep and bedrock wells are 170 to 400 feet below ground surface.

Till is a dense mixture of clay, silt, sand, gravel and rock fragments, which was deposited by glacial ice. Till has a low transmissivity and therefore does not usually produce reliable supplies of water for drinking water. Water levels in wells dug in till have a large fluctuation range and can dry up during dry seasons (Moore *et al.*, 1992).

Sand and gravel deposits were left by glacial meltwater (stratified drift), with some material being deposited by post-glacial surface water or wind-blown processes. The area on either side of the Ashuelot River was previously part of Lake Ashuelot, during the period of deglaciation. This area is at a higher elevation than glacial Lake Hitchcock, which occupied the Connecticut River valley. For simplicity sake, all sand/gravel deposits are mapped as stratified drift deposits by the USGS and the terms stratified drift and sand/gravel deposits will be used interchangeably in this section. Wells located in sand and gravel deposits can be very productive, depending on the transmissivity of the deposits.

The extent of the stratified drift deposits and an estimation of the thickness of the deposits and the transmissivity of the deposits are included in **Figure 3.3-1**. The figure shows that the transmissivity of the aquifer in the immediate vicinity of the dam, is low (less than 1,000 square feet per day) and the west side of the River in this area has thin or no sand/gravel deposits (less than 40 feet). The transmissivity of the sand/gravel deposits on the east side of the River increase from less than 1000 at the dam to up to 3,000 square feet per day, 1,250 feet upstream of the dam. A kidney-shaped area of deposits with transmissivity ranging from 3,000 to 4,000 square feet per day is located east of the River, but not adjacent to the River. The middle section of the study area could not be contoured for transmissivity and saturated thickness by the USGS. The upper section of the study area has more transmissive deposits east of the River, than on the west side. The most transmissive section adjacent to the Ashuelot River has a transmissivity range of 2,000 to 3,000 square feet per day with a thickness of 40 to 80 feet.

The storage coefficient for a sand/gravel aquifer is called specific yield. The specific yield is unit-less and represents the volume of water that can be drained per unit volume of aquifer. The specific yield values measured in southern New Hampshire ranged from 0.14 to 0.34 with an average of 0.26 (Moore *et al.,* 1992, p.29).

Natural water level fluctuations in the stratified drift aquifers are usually less than five feet, but can range up to 10 feet. Fluctuations in the till are usually larger than in the sand/gravel deposits. The elevation of the water table in the stratified drift was mapped by the USGS, with groundwater elevation contour 460 feet paralleling either side of the River and contour 480 feet showing up in some parts of the study area. The water levels were drawn to represent the summer season. The general groundwater flow direction is toward the River. Groundwater is a source of recharge to the base flow of the River.

The low flow water elevation in the Ashuelot River is approximately 455.5 feet in the vicinity of Main Street, according to the HEC analysis. The 460-foot groundwater elevation contour on the USGS stratified drift map is more than 500 feet from east side of the River in the vicinity of Main Street. The hydraulic gradient between the groundwater level and surface water level is therefore less than 0.01 (5 feet per 500 feet).

3.3.1.2 Wells

A number of sources were reviewed to obtain private well data. The identification of wells screened in the stratified drift (referred to as unconsolidated wells) were of particular interest because these wells have a potential to be affected by water level changes in the River. Conversely, bedrock wells are highly unlikely to be affected by changes in the river due to their depth and the fact that they draw from aquifers not directly in communication with the river. **Table 3.3-1** lists the unconsolidated wells located in the study area.

A number of wells registered with the State of New Hampshire were mapped using Geographic Information Systems. There were 183 wells listed in Swanzey and only eight were residential wells screened in sand/gravel deposits. The vast majority (95 percent) of wells were bedrock wells. Only two sand/gravel wells were located in the Study Area and they are listed below.

- Well NH-17188 (or Well – 712-1 or well 232.0017), located on Sawyers Crossing Road (map 41, parcel 111, owner's name listed as D. Miller), installed in 1984 to a depth of 88 feet. The well has a static water level of 22 feet and a tested yield of 12 GPM.

- Well NH-17218 (Well-232.0049) listed on Pine Street (Map 57, Lot 113, owner name Buffum) was a gravel well with a total depth of 30 feet, a static water level of 13 feet and a tested yield of 8 gallons per minute (GPM), completed in 1985.

The listings in a DES website database were also reviewed. The closest street/road names were queried. No new gravel wells were added to the study area. The nine wells identified on Spring Street were all bedrock wells, completed from 1985 to 2003. Ten wells identified on Main Street were all bedrock wells, completed from 1984 to 2003. One well listed on Prospect Street was in bedrock, completed in 1994. No wells were listed on Box Shop Road or Ashuelot Street. One well (Well-232.0049) listed on Pine Street was a gravel well, but it was the same as listed on the GIS map as Well 17218. One of six wells listed on Winchester Road was a gravel well, but this well was downstream from the study area. One of twelve wells listed on Sawyers Crossing Road was a gravel well, (identified as Well ID – 232.0017), but it was the same as Well 17188 on the GIS map. Available well logs in the vicinity of the dam that were available from New Hampshire DES were obtained.

The USGS Water Resources Investigation (WRI-92-4013) was reviewed to determine which wells within the Study Area were unconsolidated wells. Six wells were identified in the Study Area, of which five were not previously identified. The approximate locations of these wells are shown on **Figure 3.3-1**:

- Well W-87 (Latitude 425227, Longitude 721945) is a dug well with static water level 14 feet deep, completed in 1935.

Table 3.3-1 Survey of Unconsolidated Wells in the Hydrogeological Study Area

Well-ID	Other Well-ID Number	Distance From River	Assessors Map Location	Street	Latitude	Longitude	Owner	Year	Well Diameter inches	Well Type Finish/ Site	Well Depth (ft)	Well Use
From GIS Reference												
17188	232.0017	700	Map41,lot111	Sawyers Crossing			Miller, D.	1984			88	D
From NH DES website												
17218	232.0049	900	Map57,lot113	Pine St			Buffum	1985		g	30	D
From USGS Stratified Drift Report												
W87	szw87	200		Winchester St	425227	721945		1935		d / Dug		D
W65	szw65	200		Spring St	425230	721932		1950		d / Dug		D
W45	szw45	150		Winchester St	425246	721948	Miller, GC		6	p /		D
W40	szw40	200		Sawyers Crossing	425302	721917	Lehrnan, HJ		36	w / Dug		D
W41	szw41	400		Sawyers Crossing	425300	721911	Wyman, CE		4	o / Cbl		
From Unofficial Table												
U1		250	Map57,lot61	Spring St							30	
U2		257-300	Map57,lot59	Spring St							15	
U3		70	Map57, lot64	Spring St							14	
U4		275	Map57,lot77	Spring St							40	
U5		300	Map 57, lot 43	Winchester St							16	
U6		300	Map41,lot127	SawyersCrossing							49	
U7		300	Map41,lot133	Sawyers Crossing							50-75	
Wells identified as unconsolidated on USGS map, but listed in USGS Appendix A as bedrock												
W46	szw46	CLOSE	INSET A				Homestead Mill		6	x / brW		n
W85	szw85		INSET A		425218	721922		1985	6	/ brW		z
W96	szw96		INSET A		425217	721925				/brW		D
W123	szw123							1965	8	o / brW		D
W59	szw59				425251	721719		1980		o / brW		D
W62	szw62				425246	721718		1955		o / brW		D
W64	szw64	CLOSE			425304	721717		1975		o / brW		D

Notes:
Type Well Finish: G = gravel pack; P = perforated cut casing ; W = walled fieldstone or brick; O = open hole at end of casing ; d = dug well; X = open hole where casing doesn't extend to bottom (usually bedrock)
Type Site: brW = bedrock well; Wells in surficial deposits - Dug = dug well; Cbl = cable-tool well
Well Use: D = domestic; n = industrial; z = unknown

- Well W-65 (Latitude 425230, Longitude 721932) is a dug well with static water level 17.2 feet deep, completed in 1950.
- Well W-45 (Latitude 425246, Longitude 721948, owner GC Miller) had a static water level of 37 feet and a maximum yield of 11 GPM.
- Well W-40 (Latitude 425302, Longitude 721917, owner HJ Lehrnan) is a 36-inch diameter dug well with a static water depth of 3 feet.
- Well W-41 (Longitude 425300, Longitude 721911, owner CE Wyman) is a 4-inch diameter well drilled with cable tool rig.

An unofficial table of gravel wells listed in the vicinity of the dam (unknown unofficial source from DES) includes:

- Four wells on Spring Street (Map 57, Lots 59, 61, 64, 77) that have 14 to 40 feet listed as the well depth, shown on **Figure 3.3-1** as U1 through U4);
- Three wells on Sawyers Crossing Road (Map 39, Lot 2 and Map 41, Lots 127, 133) that are 9 to 75 feet deep, shown on **Figure 3.3-1** as U6 and U7); and
- One well on North Winchester Street (Map 57, Lot 43) that is 16 feet deep, shown on **Figure 3.3-1** as U5.

The summary table of the unconsolidated wells identified in the hydrogeologic study area is included as **Table 3.3-1** and the wells are located on **Figure 3.3-1**, however, the current existence and status of these wells has not been verified. It is assumed that the two wells included in the New Hampshire Department of Environmental Service's data base are in existence and have reliable data presented. The information from the USGS source may pertain to wells that are no longer in existence. The information obtained from an unreferenced table may not be reliable.

3.3.2 Hydrogeological Impacts from Dam Removal

The HEC-RAS Model of the river predicts drops in surface water levels if the dam is removed - an effect which would be particularly pronounced under low flow conditions when unconsolidated wells are most likely to fail. In the vicinity of the wells on Spring Street, Pine Street and Winchester Street the predicted drop is less than five feet during annual low flow conditions. In the vicinity of the wells on Sawyers Crossing, near where the Ashuelot River bends to an east-west direction from a north-south direction, the predicted drop is less than 4.5 feet. In the vicinity of the wells in the most upstream location of the hydrogeologic study area, the predicted drop is less than 1.5 feet. The existing water level used in the model reflects historic water levels that over-predict the current water levels, because the dam has been leaking for at least ten years. It is estimated that the difference is at least one to two feet and therefore use of the model to predict hydrogeological effects is very conservative.

The Ashuelot River and the surrounding bedrock are not considered to be in direct hydraulic communication, because low permeability till overlays the bedrock in the study area and because the productive fracture zones in the bedrock are deep. The removal of the dam and subsequent lowering of the impoundment water elevation will therefore not have a significant impact on water elevations in bedrock.

A screening level assessment was conducted to determine whether the lowering of the impoundment would have an impact on private wells finished in the sand/gravel aquifer. Two screening methods were used:

1. The downgradient extent of the recharge areas of the private wells (also referred to as capture zone) was estimated to determine if the Ashuelot River was close enough to be in the capture zone; and
2. The groundwater column in the wells were directly compared to predicted design drops in the River water level to determine if sufficient water column was available under the most stringent screening. Water columns were estimates in many cases. Therefore the relative change of the condition of sufficiency of the water column under existing and future scenarios was predicted. In certain cases, the existing water column was apparently not sufficient, as discussed below.

3.3.2.1 Screening Method 1

The contribution area for a private well is typically small because of the low rate of pumping (usually 3-12 gallons per minute). The recharge area for a well is predominantly upgradient of the well, with a small area downgradient induced back toward the well. The extent of the downgradient area can be estimated with a calculation of the distance to the stagnation point of the flow lines toward the well. The calculation assumes uniform flow to the well (a homogenous, isotropic aquifer with a fully penetrating well). The calculation is presented in **Table 3.3-2** and indicates that none of the capture zones for the wells are close to the river. The actual capture zones would likely be closer to the River than predicted, especially for wells that are not deep relative to the aquifer depth, but because the predicted capture zone extent is not close and because the pumping rate of the private wells are low, it is assumed that this method is an adequate screening method. After the dam is removed, the gradient may be steeper in the vicinity of the River, which would shorten the downgradient extent of the zone of contribution area to the wells; therefore this prediction would not change after the dam is removed.

The amount of water in the water budget for the sub-watershed area, which includes the dam, would not change with the removal of the dam. The distribution of groundwater flow upstream and downstream of the dam may be altered slightly from existing conditions, so that the flow is more evenly distributed along the River. The hydraulic gradient difference upstream and downstream of the dam would normalize after dam removal. The thickness of the upgradient aquifer (more than 80

Table 3.3-2
Downgradient Extent of Private Well Capture Zones

Well ID	Location	Max Yield Q (gpm)[1]	Transmissivity T (SF/d)	Gradient (i)	Downgradient Extent of Capture[2] (ft)	Distance from River (ft)	Is Capture Near River?
U4	Spring St	*12*	2000	0.01	18	275	No
U2	Spring St	*12*	3000	0.01	12	257-300	No
U1	Spring St	*12*	3000	0.01	12	250	No
W65	Spring St	*12*	3000	0.01	12	200	No
U3	Spring St	*12*	2500	0.01	15	70	No
W87	Winchester St	*12*	1000	0.01	37	200	No
U5	Winchester St	*12*	1000	0.01	37	300	No
17218	Pine St	8	2000	0.01	12	900	No
W45	Winchester St	11	1000	0.01	34	150	No
W40	Sawyers Crossing	*12*	1500	0.01	25	200	No
W41	Sawyers Crossing	*12*	1500	0.01	25	400	No
17188	Sawyers Crossing	12	4000	0.01	9	700	No
U6	SawyersCrossing	*12*	4000	0.01	9	300	No
U7	Sawyers Crossing	*12*	2500	0.01	15	300	No

Notes:
1. Yield values in italics are conservative estimates based on the maximum yield reported at well 17188.
2. Distance to stagnation point for the Capture Zone for private well pumping Q/(T*i*2*3.14); where Q = Maximum Well Yield Pumping Rate in cubic feet per day, T = Transmissivity in square feet per day, i = Hydraulic gradient = (460 - 455/ 500') (from USGS Stratified drift map)

feet east of the River) would not significantly change. Therefore the groundwater available in the aquifer to recharge the well would not be expected to be affected by the dam removal.

3.3.2.2 Screening Method 2

The elevation of the water table would not be expected to be altered significantly from the potential decrease in the River in the vicinity of the unconsolidated wells, but some wells do not have sufficient available water column to withstand any variation in low water elevation. Sometimes the amount of water column in the well would not be considered adequate even under existing conditions. In general, groundwater levels may vary seasonally by less than 5 feet and up to 10 feet in the sand/gravel aquifer, so a variation of less than 5 feet may be within the range of natural water table variation.

The water column measurement was based on well depth and groundwater measurements, when the data were available. In cases where groundwater measurements were not available, they were estimated, based on a measurement of the closest well. This estimate is only used for qualitative screening purposes. (See **Table 3.3-3)**

The wells were screened to determine whether it would be predicted that they had sufficient available water column under existing conditions. A subjective estimate of 15 to 20 feet measured water column needed for sufficient available water in the well is based on the need to have available water column for 5 to 10-feet of seasonal variation and 10 feet for pump drawdown, screen interval and safety factor.

Three wells (W65, W87and W40) referenced by USGS, have an existing water column of less than 4 feet. Wells W65 and W87 are old (installed in 1950 and 1935, respectively) and the measured water column of 3.6 feet and 1 foot respectively, was measured in October 1985, when groundwater levels are not necessarily at their lowest. Well W40 had only 2 feet of available water column in July 1965, but this well is 3-feet in diameter and is finished with a wall of fieldstone or brick, which is not standard for registered private wells with sanitary seals. These three wells are potentially not adequate under existing conditions and therefore were not further screened.

Four wells (U2, U1, U3, U5) were estimated to have less than 15 feet of water column and therefore perhaps do not have sufficient available water column under existing conditions. The information on these wells was from the un-referenced table and sufficient data is not available and information reliability is not known.

One well (17218) probably had more than 15 to 20 feet of measured water column and six wells (U4, W45, W41, 17188, U6, U7) had more than 20 feet of estimated water column and therefore are predicted to have sufficient available water column under existing conditions. It is noted, however, that Well W41 is constructed in a way which is not standard for modern private wells, because the casing is open at the end, instead of being finished with a well screen and gravel pack.

The amount of drawdown potentially seen in the groundwater table from a surface water drawdown would diminish with distance from the River and would vary depending on the hydraulic conductivity of the material between the River and the well. Therefore comparing the water column in a well with predicted drops in the surface water levels hundreds of feet away is very conservative.

The screening as to whether the future available water column would be sufficient was made in the same manner as existing available water column was assessed. It was assumed that there was not much of a decrease in the groundwater level at well 17218 because it is located 900 feet from the River. The seven wells that probably had sufficient water columns under existing conditions also probably had sufficient water columns under future conditions.

Table 3.3-3 - Qualitative Comparison of Water Column in the Well and Predicted Design Drop in River Level

Well ID	Street	Distance from River (ft)	Aquifer Thickness (ft)	Well Depth (ft)	GW Depth (ft)	GW Date	Existing Water Column (ft)	Exist. Water Column Sufficient?	Max. Design Drawdown	Min. Future Water Column	Min. Future Water Column Sufficient?	Change from Existing to Future?
U4	Spring St	275	80	40	*17.2*		*22.8*	yes	<5 ft	>17.8 ft	probably	borderline
U2	Spring St	257-300	80	15	*17.2*		*low*	no	<5 ft	*low*	no***	no
U1	Spring St	250	80	30	*17.2*		*12.8*	no	<5 ft	>7.8	no***	no
W65	Spring St	200	80	20.8	17.2	10/1/85	3.6	no*	<5 ft			
U3	Spring St	70	80	14	*17.2*		*low*	no	<5 ft	*low*	no***	no
W87	Winchester St	200	40	15	14	10/1/85	1	no*	<5 ft			
U5	Winchester St	300	40	16	*14*		*2*	no	<5 ft	*low*	no***	no
17218	Pine St	900	40	30	13	5/20/84	17	probably	<5 ft	15 to 17**	probably	no
W45	Winchester St	150	40	72	37	5/1/67	35	yes	<5 ft	>30	yes	no
W40	Sawyers Crossing	200	40	5	3	7/1/65	2	no*	<4.5 ft			
W41	Sawyers Crossing	400	40	40	*8*		*32*	yes*	<4.5 ft	>27.5	yes	no
17188	Sawyers Crossing	700	80	88	22		66	yes	<1.5 ft	>64.5	yes	no
U6	Sawyers Crossing	300	80	49	*22*		*27*	yes	<1.5 ft	>25.5	yes	no
U7	Sawyers Crossing	300	60	50-75	*22*		*40*	yes	<1.5 ft	>38.5	yes	no

Notes:
GW depth = groundwater depth measured on the date indicated, or estimated value in italics, based on closest well with a measurement.
Water Column = well depth - groundwater depth (or estimated depth)
Sufficient available water column assumes probably sufficient if measured value more than 15 to 19 feet; sufficient if estimated value more than 20 feet
Maximum Design Drawdown at Low Flow River = Drop in feet predicted by HEC model for low flow (90% exceedance).
*The USGS wells have suspect well construction and/or were installed more than 50 years ago (See Table 3.3-1).
* *Well 17218 is 900 feet from the river and therefore a significant change in water level is not expected
***Minimum future water column is insufficient because existing water column is insufficient

3.3.3 Summary of Potential Well Impacts

Impacts to the majority of private wells in the vicinity of the project are not expected because they are bedrock wells. The River and the bedrock are not considered to be in direct hydraulic communication, because till of low permeability overlays the bedrock in the study area and because the productive fracture zones in the bedrock are deep.

Only two unconsolidated wells registered with the State of New Hampshire were identified in the hydrogeologic study area (17188 and 17218). Twelve additional unconsolidated wells were identified in the area, but their current existence has not been verified and the limited information available for some of them, from an un-referenced table (U1 through U7), has not been validated.

In general, no direct impacts to well yields are expected in the future, because the recharge area for each well is primarily upgradient of the well. No water is being removed from the overall water budget by the removal of the dam. The downgradient extent of the recharge area for the wells is short, because of the low pumping rate of the wells and the relatively good aquifer transmissivity.

In general, a decrease in River levels of less than five feet in the vicinity of the unconsolidated private wells is not expected to impact the private well levels. The amount of drawdown potentially seen in the groundwater table from a surface water drawdown would be expected to diminish significantly with distance from the River. In addition, five feet of groundwater fluctuation is within the natural seasonal variation of groundwater levels in the sand/gravel aquifer.

Some wells may not be deep enough to withstand small variations in groundwater levels under existing or future conditions. A limited screening of available information indicates seven wells (U2, U1, W65, U3, W87, U5, W40) may not currently be deep enough to withstand seasonal variations and drawdown from pumping, with a margin of safety. Five wells (W45, W41, 17188, U6, U7) apparently have more than adequate water column available to withstand potential future drawdown or the river. Two wells (U4, and 17218) probably have sufficient water column depth. Additional information is needed to make a more definitive screening prediction.

3.4 Infrastructure

3.4.1 Thompson Covered Bridge

Among the major issues are concerns for the preservation of the Thompson Covered Bridge, an important crossing of the Ashuelot River that is listed on the National Register of Historic Places.

The scour analysis completed in 2003 indicates that contraction scour and local scour currently occur at the pier of the covered bridge in sufficient magnitude to warrant scour countermeasures. VHB performed an independent review of the Technical Report for the Thompson Covered Bridge prepared by CLD and reviewed additional information regarding scour potential obtained as part of this study. The conclusions are as follows:

- The bridge is more susceptible to scour with the dam removed. The dam removal causes additional long-term degradation of the streambed at the bridge (an additional 3 feet) and an additional 10% increase in the Contraction Scour and Local Scour depths.

- The existing pier foundation, whose base is above the lowest stream elevation, is already subject to high scour risk since it sits over 10 feet above the predicted scour depth with the dam in place. Additionally, the existing pier appears to have been partially undermined by scour and scour holes are present in other locations in the river. This is consistent with recommendations contained in the 2002 Technical Report prepared by CLD Consulting Engineers.

- Whether the dam is removed or not, scour countermeasures are recommended to protect the bridge pier and abutments. The costs of these scour countermeasures are estimated at $500,000.

A detailed discussion of our review is presented below.

3.4.1.1 Bridge Scour Analysis Background

A scour analysis was performed for the Thompson Covered Bridge in 2002 by CLD Consulting Engineers as part of a technical report for the New Hampshire Department of Transportation (NHDOT). In this report, CLD found that the masonry pier and abutment foundations are susceptible to scour (calculated scour depth below the foundations) whether the Homestead Dam, located approximately 200 feet downstream, is removed or left in place. CLD discussed several scour countermeasure alternatives and recommended reconstructing the center pier with a deeper footing and monitoring the abutments and the adjacent bank slopes.

As part of Homestead Dam Removal Feasibility Study, VHB was charged with:

- Reviewing the 2002 CLD technical report and other available information in order to determine if additional scour analysis is warranted.
- Recommending an appropriate scour countermeasure plan that considers the conceptual alternatives presented in the CLD technical report as well as other practical alternatives.
- Developing an order of magnitude cost for the recommended scour countermeasure(s).

3.4.1.2 Long-term Degradation

The effect of long-term degradation of the channel after dam removal was assumed to be negligible at the bridge in the CLD technical report. However, comparing the channel cross-section from the Flood Insurance Study (1980) taken over 25 years ago with the channel cross-section taken at the bridge in 2004, the channel thalweg elevation (approximately elevation 449.9) has lowered approximately 1.2 feet (elevation 448.7) since the late 1970's. Part of this degradation may be attributed to the deterioration and leakage of the dam as it continually lowers the headpond elevation upstream. As discussed in the geomorphological analysis in Section 3.2 above, an additional three feet of headcut or degradation is likely to occur at the bridge if the dam and the buried causeway immediately upstream of the dam are removed without appropriate stream bed stabilization. This would add about four to five feet to the total scour depth estimated in the CLD report (report was based on FIS channel cross-sections for scour calculations.)

3.4.1.3 Contraction Scour

The effect of contraction scour before and after dam removal is similar for the 100-year event according to the CLD technical report (approximately one foot). However, the greatest scour potential will likely occur for a larger flood event. A 500-year event "check flood" is typically analyzed in order to estimate a maximum scour depth and to design an appropriate countermeasure. Based on the revised hydraulics analysis performed as part of the Dam Feasibility Study, it does not appear that the contraction scour estimated by CLD would likely increase more than a foot or two for such an event depending on bed material (riprap or natural alluvium).

3.4.1.4 Local Scour - Piers

The effect of local scour (at the abutments and piers) before and after dam removal is similar for the 100-year event according to the CLD technical report. Again, a 500-year event "check flood" should be analyzed during final design to estimate a maximum scour depth and to design an appropriate countermeasure.

The effective pier width assumed in the CLD report was five feet. However, the existing pier is wider at the bottom of the channel. If a reconstructed foundation is placed, an effective pier width of about eight feet is likely (assuming the masonry pier base is about 7 feet wide). This parameter will increase computed pier scour depth from the 8.3 feet noted in the CLD report to over 11 feet. If a stone protection countermeasure is properly designed and placed, the estimated pier scour depth could decrease by more than 50 percent.

3.4.1.5 Local Scour - Abutments

The equations currently used to estimate abutment scour result in conservative scour depths. Although the pier scour equations can be modified to account for riprap protection, the equations for abutment scour cannot. Until there are more reliable equations for abutment scour, standard practice is to use engineering judgment and ensure that abutments are designed to accommodate contraction scour, channel migration, and long-term degradation.

3.4.1.6 Scour History

From historical accounts, it appears that a dam at this site existed prior to the construction of the Thompson Covered Bridge in 1832. Large, damaging floods have occurred several times along the Ashuelot River although there is little information prior to 1900. The flood of record occurred in Mach of 1936 with an estimated discharge of 13,400 cfs (nearly double the estimated flow for the 100-year flood). This clearly indicates that the existing bridge has been subjected to sizeable floods during its life time. However this does not mean that the bridge is safe for all future floods at or below that level. Minor meanders in the stream channel, debris carried by a flood event, or fallen trees can change the direction of flow near a pier and result in significant scour in a short period of time.

Borings at the bridge were taken as part of the CLD technical report. The boring information provides evidence that the bed material supporting the upstream portion of the pier has likely scoured more than two feet below the bottom of masonry pier (elevation 449.2). This likelihood is substantiated by the visible differential settlement of the pier masonry courses (approx. 4 inches as noted in a 1991 bridge inspection report). Additional evidence of pier scour is mentioned in an underwater inspection report from 1992. Inspectors observed large voids at the apparent bottom of the pier extending completely across the pier (six to eight feet across). Riprap was later "heaped" around the piers and placed in front of the abutments during a rehabilitation project in 1993.

The Cresson Covered Bridge at Sawyers Crossing is a similar two-span bridge constructed in 1859 located approximately 4 miles upstream. Bridge inspection reports indicate heavy scour activity and debris accumulation at the center masonry pier resulting in pier masonry settlement of almost one foot. This bridge was rehabilitated in 1996 however, minor scour (exposing top of stone footing) and heavy drift debris was still noted in a 1999 underwater inspection report.

3.4.1.7 Scour Countermeasures

Based on review of the CLD technical report and other available information, scour countermeasures are required at the Thompson Covered Bridge whether the dam downstream is removed or left in place. The final countermeasure that is selected should be designed based on an additional scour analysis that considers the following:

- A 500-year "check-flood" event in order to estimate maximum flow velocities and water depths.
- Any channel armoring that can justifiably be accounted for in resisting scour.
- Arresting channel degradation in the vicinity of the bridge assuming the dam is removed or breached.
- The historical importance and value of the bridge.
- The level of risk with respect to the cost of the scour countermeasures.

There is enough information available to suggest a likely scour countermeasure plan and develop order of magnitude costs. A final scour countermeasure solution is an interdisciplinary effort that will require input from a competent geotechnical engineer that is familiar with scour and bridge foundations.

Several possible scour countermeasures are as follows:

Do Nothing and Monitor Only

This option is typical for bridges that are at "low risk." Since the channel bottom is more than 6 inches below the pier base (measured less than eight feet away from the face of the pier) and there is evidence of pier scour still occurring, the bridge pier is at "high risk." Based on the degradation that has occurred over the past 25 years and the estimated degradation associated with dam removal, the embankments around the masonry abutments are at risk unless the existing riprap is removed, redesigned, and constructed properly. This option is not recommended.

Installing Soil Containment Piling

This option involves driving or drilling piling in order to prevent soil displacement on one side of piling perimeter. Based on the dense gravel till encountered in the borings, piling installation would likely require drilling or pre-drilling. Unless the bridge is temporarily moved, installing piling beneath the bridge and around the existing foundation would require special construction equipment to accommodate the limited clearance under the bridge. This option is not recommended.

Foundation Reconstruction

This option involves shoring the existing bridge, installing water diversion structure(s), carefully removing the existing pier masonry, constructing a new foundation, and rebuilding the masonry pier stem. A deep pile or shaft foundation will require special low clearance equipment and construction methods or moving the existing bridge out of the way. A relatively shallow foundation (approximately 6 to 8 feet below the stream bed) could be constructed with traditional shoring, water diversion, and construction methods without moving the bridge. A reconstructed shallow foundation is recommended at the center pier provided a properly designed and constructed keyed stone fill is placed and the computed pier scour is less than 6 to 8 feet (considering the riprap armor).

Grouting

This option involves drilling a relatively small rod through the soil and injecting grout under pressure to form a grouted pile column. Based on the dense gravel till encountered in the borings this option may be difficult to construct properly. This option is also very expensive in comparison to traditional footing construction or piling construction. This option is not recommended.

Streambed and Embankment Armoring

This option involves installing water diversion structures, excavating to the depth of armoring, placing a geotextile fabric (specifically designed for the application), placing a small relatively small lift of granular material on top of fabric, and installing a properly graded and compacted keyed stone fill with minimal voids. This option is recommended whether the dam is removed or left in place and whether the pier is reconstructed with a deep of shallow foundation.

Superstructure Capacity and Strengthening Evaluation

This option involves increasing the bridge's capacity to support itself and any superimposed loads (such as vehicles) if the center pier were to completely fail. The cost of constructing a support system that maintains the historical nature of the bridge is very high compared to armoring and pier foundation reconstruction. This option is not recommended.

3.4.1.8 Scour Countermeasure Recommendations

Although the 2002 CLD Technical Report contained some debatable assumptions for determining the total scour at the bridge, it is likely that a final scour analysis will support their countermeasure recommendation for the pier; reconstruct with a new mass concrete footing to an elevation of about 442. However, the degradation and contraction scour must be mitigated for in order for this countermeasure to provide a cost-effective and "low risk" assessment for pier and abutment scour. A properly designed and constructed channel armor is recommended to resist total scour in combination with a reconstructed pier.

Based on new information indicating that the stream bed is below the pier in the vicinity of the pier and that the riprap at the pier does not appear to be well graded or properly placed (at the level of the stream bed), the pier is at high risk for scour whether the dam downstream is removed or not.

The greatest potential for significant local pier scour is during relatively large floods when there is a significant amount of sustained stream force. If such a flood were to occur, it is possible that the dumped stone around the piers could be undercut along the perimeter, causing them to move and settle. If the voids between the settled stones are large enough, the vortices will likely undermine the bed material and this could occur fairly quickly, especially if the channel bottom is currently below the bottom of the pier. The magnitude of the flood required to completely scour the pier and when this flood occurs is unknown. However, a degrading channel is a continual process that occurs over many floods (small and large) and this also compromises the effectiveness of the existing riprap.

The presence of the dumped stone around the pier will reduce scour potential directly beneath the stones as indicated by the fact that the existing pier appears to have been in place for over 170 years. However, given the importance of this bridge to the community and historical significance, the scour countermeasures recommended for this bridge should be programmed for funding whether the Homestead Dam is removed or not.

3.4.1.9 Estimated Scour Countermeasure Costs

The major construction components and estimated construction costs associated with the suggested scour countermeasures are described below:

Temporary Superstructure Shoring
Temporarily close bridge (if maintenance of traffic permits), install temporary bents on each side of the center pier to support the truss self-weight.

Channel Diversion/Dewatering
Temporarily lower the headwater elevation and or remove the existing dam and causeway. Route the river under one span of the bridge with water diversion methods. Reroute the river under the other span once pier reconstruction and armoring is completed in the first phase. Remove water diversion devices once channel work is complete or construction is above the water level.

Install Keyed Stone Fill Bank and Channel Armor
Excavate existing channel and bank material upstream and downstream of bridge (50 feet up and downstream assumed. Existing abutment ripap extends over total length of about 55 feet at each bank). Install geotextile, granular bedding material, and keyed stone fill. Extend fill from bank to bank (terminating the stone fill at the toe of slopes with a 10 foot apron and extending the stone fill a distance of 2 x the pier scour depth each side of the pier, nearly encompasses the entire channel width) to provide lateral confinement of the armoring and to mitigate for degradation. Thicken and key flank edges on the upstream and downstream edges to control local undermining and settlement adjacent to the natural stream bed.

Remove and Reconstruct Pier with New Foundation
Carefully dismantle masonry pier. Excavate to proposed footing depth (shallow support of excavation devices may be required). Form and place new mass concrete footing. Reconstruct masonry pier, backfill and hand place armoring around pier as required.

Table 3.4-1
Bridge Scour Countermeasure Cost Summary

Item	Cost
Remove and Reconstruct Pier with New Foundation	$85,000
Install Keyed Stone Fill Bank and Channel Armor	$200,000
Channel Diversion/Dewatering	$30,000
Temporary Superstructure Shoring	$50,000
Mobilization Costs (10% Assumed)	$40,000
Contingencies (25% Assumed)	$95,000
Estimated Construction Cost Total	**$500,000**

3.4.2 Fire Fighting Water Supply

The Swanzey Fire Department was consulted to assess the current use and value of the impoundment for fire fighting. Currently, two known hydrants are located in the impoundment. Both are located in the immediate vicinity of the dam, on each side of the river.

On the east side of the river, directly adjacent to the West Swanzey Fire Station, is a hydrant and an electric pump station that was used in the past as a fire fighting water supply. The hydrant intake is located in the bed of the river adjacent to the Thompson Covered Bridge. Attempts to locate the intake structure during site surveys were unsuccessful. This hydrant is reported to have been inoperable for several years and there are no immediate plans to repair it (Sly Karasinski, Town of Swanzey Fire Department, personal communication, 2004). The hydrant is not a priority for the Fire Department because of the 1,000 gal/min fire pump located across the river at the mill. The existing hydrant also has a small diameter pipe, limiting its pumping capacity, which makes it a lower priority hydrant to continue to maintain.

On the west side of the river is a pump station with an intake in the impoundment that serves to pressurize the sprinkler system of the Homestead Mill building and is accessible to the Fire Department. Property managers at the Homestead Mill exercise this system monthly to maintain its usefulness (Doug Brown, personal communication, 2004). Unfortunately, there are no known plans of the pump house or water intake and attempts to locate the intake during site surveys were unsuccessful. The date of the construction of the intake is unknown; however, it is very likely that it was placed after the construction of the dam. Therefore, the elevation of the intake will likely need to be adjusted if the dam were to be removed. Relocation of the intake is a relatively straight forward procedure that could be accommodated during final design if the dam removal option is pursued.

Other fire fighting resources in the study area are worth noting as well. First, there is a 15,000 gallon cistern at the community church near the West Swanzey Fire Station. The integrity of the cistern is questionable, and it would likely require repairs to be fully usable. The cistern could be used to provide additional storage of water that fire fighters could draw from in an emergency. Additionally, while there are no other hydrants in the study area, the fire department has two other points at which they can access the river by pump truck. One of these sites is located at the Sawyers Crossing/Route 10 Intersection, and the other is at the Cresson Bridge. These accesses were constructed for a 10-foot pump lift. Any greater lift caused by a drop in river surface elevation would be a cause for concern. Note that the HEC-RAS analysis reported in **Section 3.2.2** and **Table 3.4.2** illustrates that the river will drop only about six to fifteen inches under typical flows.

Table 3.4.2 Water Surface Elevations at Sawyers Crossing Fire Access

		Water Surface Elevation (fsl)			
Flow Recurrence	**Flow (cfs)**	**Dam In**	**Dam Out**	**Drop (ft)**	**Drop (in)**
90% Exceedance	60	455.22	454.00	1.22	15
July Mean	180	455.79	454.81	0.98	12
June Mean	400	456.78	455.93	0.85	10
Annual Mean	520	457.25	456.49	0.76	9
May Mean	750	458.05	457.40	0.65	8
Bankfull	2,600	462.40	461.88	0.52	6

Notes: cfs = cubic feet per second, fsl = feet above sea level

Even under the driest conditions, it is predicted that the river will be more than two feet deep in this vicinity if the dam were to be removed versus about 3.5 feet under existing conditions. Thus, the removal of the dam would not substantially affect the Fire Department's ability to collect water in an emergency situation.

3.4.3 USGS Gauging Station

The USGS owns and operates a gauging station on the Ashuelot River, approximately 15 feet west of the Homestead Woolen Mills Dam, on the western side of the river. The City of Keene cooperates with USGS to maintain the gauge, as they use it to monitor their waste load allocation for their waste treatment facility. The stream gauge was established during the fall of 1994, with records dating to April 1, 1994. Daily flow data, including real-time measurements, are currently available from this station.

The gauge consists of a Sutron 8210 data collection platform (ID DD1AF1F8) with modem capabilities wired to an Accubar pressure transducer with a purge/bypass conoflow, air temperature thermistor, and a tipping bucket rain gauge. It is housed in a 4 ft x 4 ft plywood house. The Sutron is powered by a single battery with a 10 watt solar panel.

Dam removal will clearly impact the gauging station. As discussed above, the river bed profile and cross section would be reshaped substantially. Water surface elevations would drop substantially. These changes will affect the stage-discharge relationship currently calibrated for the gauge. This will affect the ability to collect, record, and transmit accurate stream flow data. However, due to the poor condition of the dam, water currently leaks from both the dam's base and over its top. Such leaks affect the river stage above the dam. Thus, the accuracy of the stream gauge is already affected by the poor condition of the dam.

Two options are suggested for re-locating the stream gauge if the dam is removed.

1. Locate a natural bedrock controlled riffle upstream of the current gauge location to reposition the existing gauge structure and equipment. The closer the gauge is sited to the City of Keene, the better it will serve one of its designated purposes for use in allocating waste loads for the City of Keene. Following installation, the gauge would require recalibration. The estimated cost of this option is approximately $5,000.

2. Following dam removal, reposition and recalibrate the stream gauge at the existing site. The estimated cost of this option is approximately $2,000.

The USGS in Pembroke, NH, was consulted regarding the potential impacts to the gauge. USGS recommended option one if dam removal is to occur (Kenneth Toppin, USGS Section Chief, personal communication). USGS also indicated that, regardless of whether the dam is removed, relocating the gauge upstream would provide better data integrity because of the stage-discharge relationship problem created by the dilapidated condition of the dam.

3.4.4 Recreational Resources

There are no developed recreational facilities or formal public access areas within the reach of the Ashuelot River between West Swanzey Village and the upstream extent of the impoundment. However, informal access to the river occurs along both banks in several areas throughout the reach. These access points tend to correspond with residential areas such as along the Spring Street neighborhood on the east side of the river in West Swanzey.

More significant informal public access to the river occurs in two locations. The West Swanzey Athletic Association property just north of the Thompson Covered Bridge provides parking and a relatively low, flat bank that was observed to be used for launching of car top boats on several occasions during this study. And, a small picnic area and informal canoe access is located on the west side of the river directly upstream of the Cresson Bridge.

Flat water canoeing is very popular on this reach of the Ashuelot River. A local canoe club sponsors an annual canoe race on the river from Keene to West Swanzey each May. The event is sanctioned by the US Canoe Association and is heavily attended by canoe enthusiasts from around the northeast. Any substantial drop in the river depth or width could pose a threat to the continued success of this flatwater race. Below, **Table 3.4-3** shows river characteristics for selected locations under typical May flow conditions. These data will help allow assessment of the magnitude of potential impacts. (See also the data in **Appendix E**.)

Table 3.4-3 Water Surface Elevations and Depths under Average May Flows (Q = 750 cfs)

	Channel Depth (ft)			Channel Width (ft)		
Location	Dam In	Dam Out	Δ (ft)	Dam In	Dam Out	Δ (ft)
3000 ft N Keene/Swanzey	6.45	6.42	0.03	75.18	74.96	0.22
Cresson Bridge	6.35	5.70	0.65	144.20	139.94	4.26
Indian Fish Weir	7.34	6.12	1.22	160.91	129.01	31.90
Meander Bend (~1 mile upstream)	12.79	10.16	2.63	133.79	130.32	3.47
Spring St. terminus (~1/2 upstream)	14.08	10.25	3.83	164.00	158.11	5.89
Thompson Covered Bridge	7.51	2.45	5.06	115.00	82.23	32.77

Notes: Water surface elevation represents the elevation of the river's surface in feet above sea level. Channel depths are maximum values for the cross-section at each location.

From the table, it becomes readily apparent that the removal of the Homestead Dam would have a diminishing effect on the canoe race as one moves upstream of the dam. In the vicinity of the start of the race in Keene, for example, there would be no detectable effect under average May flows and the race would be able to occur as usual. However, as one approaches West Swanzey, the effect of the dam removal would become more apparent. In the vicinity of the Cresson Bridge, for example, average channel depths in May are predicted to drop from 6.35 feet to 5.70 feet and the width of the open water is expected to decrease from 144.20 feet to 139.94 feet. While this is an appreciable change, it would have a relatively inconsequential effect on canoeing. Closer to the actual dam site, the predicted drop in water levels is greater, but average depths are also greater due to the river geometry. Thus, until one approaches the actual dam site, the changes are unlikely to prevent navigation by canoe in the late spring or early summer.

3.5 Cultural and Historic Resources

Any replacement or removal of the Homestead Dam would likely involve the use of Federal funding, as well as permits from the US Army Corps of Engineers. Because Federal funding or permitting is involved with such an undertaking, the project would be subject to review under Section 106 of the National Historic Preservation Act (36 CFR 800). Section 106 requires Federal agencies to take into account the effects of their undertakings on properties listed in or determined eligible for listing in the National Register of Historic Places (National Register), explore alternatives to avoid or minimize harm to historic properties, and consult with State Historic Preservation Officers (SHPOs) to resolve any adverse effects to historic properties resulting from the undertaking. In New Hampshire (NH), the Director of the NH Division of Historical Resources (DHR) is the SHPO.

The potential removal or replacement of the Homestead Woolen Mill Dam has generated a number of concerns related to the preservation of cultural and historic resources. During early consultation between the NH Department of Environmental Services (DES) and DHR on the Homestead Dam Feasibility Study, DHR requested an assessment of the historical and architectural significance of the dam and the associated Homestead Woolen Mill complex. DHR also expressed concern about the effect removal of the Homestead Dam would have on the historic integrity of the surrounding West Swanzey Village Historic District. DHR further requested a study to identify known and potential archeological resources upstream and downstream of the present dam impoundment that could be impacted by changes in the hydrology of the Ashuelot River resulting from dam removal. Both Swanzey residents and DHR were concerned about the effect dam removal would have on the structural soundness of the adjacent West Swanzey (Thompson) Covered Bridge, which crosses the dam impoundment area.

As part of the Feasibility Study for the Homestead Dam, VHB, in collaboration with Victoria Bunker, Inc., identified potentially significant cultural resources in the vicinity of the dam and evaluated the historical and architectural significance of the dam structure within the context of the West Swanzey Village Historic District. This section describes the results of those studies and the potential impacts each alternative would have on significant cultural resources.

Assessments of the effects of the proposed alternatives on cultural resources were made utilizing the methods outlined in the Federal regulations for the Section 106 review process (36 CFR 800). According to the Section 106 regulations, an adverse effect to an historic property is found when a Federal undertaking "…may alter, directly or indirectly, any of the characteristics of a historic property that qualify the property for inclusion in the National Register in a manner that would diminish the integrity of the property's location, design, setting, materials, workmanship, feeling, or association."[2] Examples of activities resulting in adverse effects under Section 106 applicable to this project include:

- Physical destruction of or damage to all or part of a property;
- Alteration of a property, including restoration, rehabilitation, repair, maintenance, and stabilization that is not consistent with the Secretary's Standards for the Treatment of Historic Properties (36 CFR part 68) and applicable guidelines; and
- Change of the character of the property's use or of physical features within the property's setting that contribute to its historic significance

3.5.1 West Swanzey Village Historic District

DHR determined the Village of West Swanzey eligible for the National Register as an historic district in 1995. At the time of the determination of eligibility, contributing resources and specific bounds in the historic district had not been determined. In response to DHR's request for an evaluation of the historical significance of the Homestead Woolen Mill complex, VHB completed a DHR Historic District Area Form for the Village of West Swanzey to identify contributing properties and recommend district boundaries (**Appendix A**).

The Homestead Dam and its associated mill buildings are situated at the center of West Swanzey, one of five village settlements in the Town of Swanzey. Settled in 1736, West Swanzey was the site of the first water-powered grist and sawmills in the town, which were located in the vicinity of the Homestead Dam. Beginning in the 1830s, West Swanzey developed large-scale woodenware and woolen manufacturing interests, which remained the main economic drivers of the community until the early 20th century. At the peak of industrial production in the 1880s, the village contained a woolen mill, two wooden box companies, two wooden ware mills

2 36 CFR Part 800 -- Protection of Historic Properties; §800.5 Assessment of Adverse Effects

manufacturing buckets and pails, and a grist mill. The influx of capital from the mills contributed to the construction of significant religious, civic, commercial, and residential buildings in the village, clustered between the manufacturing interests on the Ashuelot River on the west, and the Ashuelot Railroad on the east.

Completion of the DHR Historic District Area Form determined that the Homestead Woolen Mill and the associated timber crib Homestead Dam were important contributing resources in the West Swanzey Village National Register Historic District because of the historic and architectural significance of the mill property. The Homestead Woolen Mill was the longest continuously operating manufacturing interest in the history of West Swanzey, and contains significant industrial buildings and structures reflecting over a century of architecture and technology associated with woolen manufacturing in the region. These include the ca. 1860 timber crib Homestead Dam, the ca. 1866 wood frame Stratton Woolen Mill, a portion of the 1868 brick Stratton Mill, the ca. 1930 brick boiler house, and the ca. 1950 brick mill building.

Each of the proposed alternatives for removal or replacement of the Homestead Dam would result in an adverse effect under Section 106 to the National Register-eligible West Swanzey Village Historic District, as they involve physical destruction of all or part of a contributing resource in the district. Removal or replacement of the Homestead Dam with a modern structure, even of similar design, would diminish integrity of setting, design, materials, workmanship, feeling and association in the historic district. If an alternative involving removal or replacement of the Homestead Dam was chosen, NOAA, DES, and DHR would have to consult to find ways to avoid, minimize, or mitigate the adverse effect to the National Register-eligible West Swanzey Village Historic District.

Alternative E, which proposes adding hydropower to the existing dam, has the potential to cause an adverse effect under Section 106 through alteration of the dam. Alternative E would need to be further developed to determine the nature of the necessary alterations to the dam structure before a definitive assessment of effect could be made.

3.5.2 West Swanzey (Thompson) Covered Bridge

The West Swanzey Covered Bridge, also known as the Thompson Covered Bridge, was constructed in 1832 to carry Main Street over the Ashuelot River. The bridge is situated immediately upstream of the Homestead Dam and has a central pier resting in the dam impoundment. The West Swanzey Covered Bridge was listed on the National Register in 1980 for its significant association with the history of transportation in Swanzey and the State of New Hampshire. The bridge is also a contributing resource in the National Register-eligible West Swanzey Village Historic District.

Residents of Swanzey and DHR expressed concern about the effect dam removal and the resulting increased water flow would have on the structural integrity of the West Swanzey Covered Bridge. According to the bridge scour analysis presented in Section 3.4.1, removal of the Homestead Dam has the potential to adversely affect the West Swanzey Covered Bridge under Section 106 by damaging all or part of the structure. Removal of the dam could lower the streambed in the vicinity of the bridge by as much as three feet, which could in turn result in undercutting of the bridge pier and abutments. Dam removal would also increase the existing scour on the bridge substructure, which already warrants countermeasures, by 10 percent. Dam replacement with fish passage and the addition of hydroelectric power to the existing dam would not accelerate scour at the West Swanzey Covered Bridge, however, the issue of elevated scour under existing conditions would remain.

NOAA, DES, and DHR would have to consult on ways to avoid, minimize, or mitigate potential adverse effects to the West Swanzey Covered Bridge resulting from dam removal. The scour analysis presented in Section 3.5.1 identified several ways to avoid and minimize potential damage to the bridge. These include stream bed and embankment armoring and pier foundation reconstruction.

3.5.3 Archeological Resources

At the request of DES and DHR, Victoria Bunker, Inc. completed a Phase 1A Archeological Reconnaissance-level Survey to define concerns for archeological resources within the immediate vicinity of the Homestead Woolen Mill and Homestead Dam, as well as a 5-mile reach of the Ashuelot River (study area). The study investigated Euro-American and pre-contact Native American archeological resources, with particular attention to the Swanzey Fish Dam, a Native American structure submerged upstream in the dam impoundment. The study discussed existing and expected archeological resources within the study area. Because no subsurface field investigations were conducted, the study does not identify the presence, boundaries, age, function, or integrity of sites within archeologically sensitive zones. The study did not compile sufficient data to assess National Register eligibility for known archeological sites or features. Further study will be required to determine site presence within archeologically sensitive areas and the National Register eligibility of known and newly identified archeological resources. Because further study is required, only a preliminary assessment of potential impacts to archeological resources is offered.

The Phase 1A survey found that the study area contains pre-contact Native American and historic Euro-American archeological remains and areas sensitive for such remains in terrestrial and submerged settings. Known archeological sites indicate a long Native American occupation in the area dating from 10,000 B.P. to the Contact Period (450-300 B.P.). Six sites dating from Native American inhabitation of the area are within the project study area, including the Swanzey Fish Dam. Though Native American fish dams are common in North America, the Swanzey Fish Dam is a significant feature in that it is the first such structure documented in New England. The Phase 1A survey found that pre-contact Native American sites have survived along the Ashuelot River despite modern growth and development, and asserted that it is reasonable to expect to discover additional sites even in heavily developed or modified areas. The survey report assigned archeological resource sensitivity for the occurrence of Native American sites, artifacts, and features to all undisturbed terraces along the Ashuelot River within the study area.

Though there are no recorded historic archeological sites within the study area, the continuum of Euro-American industrial and agricultural use within the study area is also likely preserved archeologically. Archeological remains of historic period sites are expected to be related to domestic, transportation, or industrial activities dating from between the 1750s and 1900. The area immediately surrounding the Homestead Dam also has strong potential to provide information on historic industrial development such as structural remains and artifacts.

The Phase 1A survey report recommended continued archeological survey for the portions of the study area that could be impacted by the proposed alternatives. Further study would include a Phase 1B site identification study for areas with resource sensitivity, and intensive survey of known sites to determine the extent of the resources' area of potential effect. Phase 2 investigations were recommended for any archeological resources discovered during the Phase 1B site identification study to determine whether the resources were eligible for the National Register. The Phase 1A survey report further recommended that the Swanzey Fish Dam not be exposed until a plan for research, recordation, and sampling is developed in consultation with DHR.

All of the proposed action alternatives have the potential to adversely affect significant archeological resources within the study area under Section 106. The impacts of primary concern are exposure of submerged archeological resources and areas of archeological sensitivity, the potential for erosion of archeological sites and archeologically sensitive areas as a result of increased water flow speeds after dam removal, and ground disturbing activities associated with dam removal, dam replacement, and construction of a fish bypass channel.

Hydraulic and geomorphic analyses presented in Sections 3.2 and 3.3 indicate that the removal of the Homestead Dam would decrease the impoundment volume by 30 percent, with an 8 percent decrease in impoundment surface area, and a 24 percent decrease in depth of the stream under annual mean flow conditions. Removal of the dam would not, however, substantially decrease stream width, but would increase velocities. This predicted increase in velocities poses a concern for the integrity of archeological resources found along the river banks.

Substantial concerns have been expressed by members of the community and by DHR over potential impacts to the Swanzey Fish Dam. **Table 3.5-1** summarizes data from the hydraulic analyses for a cross section at the fish dam. Generally, changes in depths and velocities under a dam removal scenario are not likely to be perceptible in this location under higher flows. Even under mean annual flows, the maximum

Table 3.5-1
Depth, Velocities, and Tractive Force at the Swanzey Fish Dam, Ashuelot River

		Max. Stream Depth (ft)		Velocity (fps)		Tractive Force (kg/m^2)[2]		Incipient Diameter (Soil Classification)	
Flow Condition	**Q (cfs)[1]**	**Existing**	**Dam Out**	**Existing**	**Dam Out**	**Existing**	**Dam Out**	**Existing**	**Dam Out**
90% Exceedance	60	5.09	2.19	0.15	0.53				
July Mean	180	5.51	3.33	0.40	0.84				
June Mean	400	6.27	4.65	0.73	1.14				
Annual Mean	520	6.65	5.20	0.87	1.25				
May Mean	750	7.34	6.12	1.09	1.42				
Bankfull	2,600	11.39	10.59	2.04	2.26	0.64	0.82	fine gravel	fine gravel
2-Year Flood	2,940	11.94	11.19	2.16	2.37				
10-Year Flood	4,630	14.23	13.63	2.65	2.82				
50-Year Flood	6,190	15.91	15.44	3.01	3.15				
100-Year Flood	6,840	16.54	16.12	3.14	3.26				

Notes:
1. "Q" denotes the flow in the river under various categories in cubic feet per second. These flows were determined primarily from stream gauge data maintained by the USGS.
2. Tractive force analysis was completed only for the bankfull flow (approximately 2,600 cfs or the flow expected approximately once in every 18 months).
3. "Incipient diameter" is the diameter at which individual particles subjected to a shear stress begin to move based on the tractive force analysis. Here, we report the value according to the USGS soil classification scheme.

depth of the river at the fish dam would be expected to drop from 6.65 feet to 5.20 feet, and velocities increase from 0.87 fps to 1.25 fps. The tractive force analysis (see Section 3.2.3 for a detailed discussion) indicates that, while tractive forces will increase under the dam removal scenario, the actual increase is not expected to be significant.

These results suggest that the potential for damage to the fish dam is slight but can not be entirely ruled out. The historical significance of the fish dam requires that care be taken to ensure that it is not unduly impacted if the dam removal alternative is pursued. Careful planning and study will be required to ensure proper preservation of the feature.

Ground disturbing activities associated with full dam removal, dam replacement with the construction of fish passages, dam removal and replacement with a rock fish ramp, and the addition of hydropower to the existing dam all have the potential to adversely affect archeological resources in the immediate vicinity of the dam. These resources include those that may provide information on the continuous historic industrial development on the banks of the Ashuelot River in West Swanzey from 1736 to the present. Because there has been only reconnaissance-level investigation of archeological resources in the vicinity of the Homestead Dam, more investigation would be required to definitively assess the effect ground disturbing activities associated with dam removal or replacement would have on significant archeological resources.

3.6 Natural Resources

3.6.1 Fisheries

3.6.1.1 Restoration Efforts

One objective of removing the Homestead Dam is to eliminate the obstacle posed by the dam and thus provide fish passage for anadromous and catadromous fish. The dam is located in a reach of the river that nearly bisects suitable spawning and nursery habitat for species such as Atlantic salmon *(Salmo salar),* American shad *(Alosa sapidississsma)* and blueback herring (*Alosa aestivalis*) (the later two species are sometimes referred to as "alosids" or "clupeids"). Fish passage is required for these species to connect riverine habitat required for their reproduction, nursery and juvenile habitat to their adult habitat. Below, we discuss the diadromous fish species present in the Ashuelot and consider whether removal of the dam is likely to meet the objective of providing upstream fish passage above West Swanzey.

It is important to note that many migratory species of fish inhabited the Ashuelot River system in the past (see **Table 3.6-1**), but dams, habitat degradation and overfishing have eliminated important species such as Atlantic salmon and reduced populations of other species to a small fraction of their previous numbers. In addition to Atlantic salmon, shad and herring are other species targeted for restoration to the Ashuelot. These clupeid species were never lost entirely from the Connecticut Basin, which increases the chances that genetically-native populations can be re-established in the Ashuelot. Although species such as the sea lamprey, eel, and herring are perceived to have minimal recreational values, they do play important roles in a healthy, ecologically strong river system.

Table 3.6-1
Migratory Fish Species in the Ashuelot River

Species	Restoration Status
Atlantic salmon	Under restoration beginning in spring 1995
American shad	Under restoration beginning in spring 1998
Blueback herring	Under restoration beginning in spring 1999
Sea lamprey	Not targeted at this time
American eel	Not targeted at this time

Source: Sprankle (2000)

The damage to these native fish populations is documented in comprehensive biological surveys of the Connecticut River Watershed conducted in the late 1930s. These reports brought together historical information that documents the presence of salmon as well as American shad and river herring in the Ashuelot and their use by earlier settlers and Native Americans. Loss of these native populations was noted as early as 1798, when dams on the mainstem of the Connecticut River were constructed, which prevented these fishes from reaching their home spawning and nursery habitats in the waters of New Hampshire and Vermont.

Since the late 1960s, a cooperative effort of the USFWS, the NHF&GD and other state fishery agencies and interested non-profits have been working to restore and enhance native diadromous fish populations to the rivers of the Connecticut Basin. Atlantic salmon has largely been the focus of this long-standing partnership; however the clupeids are also targeted for restoration.

Current restoration strategies begin with assessment of river segments to find appropriate fish habitat, which helps to determine stocking rates. Young fish are either raised in hatcheries (generally the case for salmon) or are collected from downstream locations (more typical for clupeids), then stocked into these habitat units through the efforts of the USFWS and the NHF&GD. These efforts have had some successes, but clearly additional work is needed to develop sustainable populations in both the mainstem Connecticut and its tributaries such as the Ashuelot.

In the largest clupeid habitat survey of the Ashuelot completed to date, Sprankle (1999) identified approximately 94 hectares of spawning and nursery habitat from the Surry Dam to the Kelly Farm in Winchester and estimated that an additional 45 hectares existed from Kelly Farm to the confluence with the Connecticut and in the lower reaches of the South Branch of the Ashuelot (See **Appendix G**.) From these habitat surveys, Sprankle (1999) estimated that the Ashuelot River could produce an annual run of more than 11,500 shad and 47,000 herring. **Table 3.6-2** details stocking activity for clupeid species in the Ashuelot Basin since 1998, when the stocking effort began.

Even greater numbers of salmon are stocked into the Ashuelot. In 2004, a total of 5.30 million Atlantic salmon fry were reared for stocking in the Connecticut River watershed. Of that total, 1.21 million were stocked in the main stem and tributaries within New Hampshire's boundaries. Approximately 22,000 of these fish were stocked into the South Branch of the Ashuelot River, a number similar to stocking levels reported for 2003 (NHF&GD unpublished grant reports F-50-R-19 and F-50-R-20).

Note that these salmon and clupeid fish are "natal homing" animals. That is, when the fish become mature, the fish migrate downstream to the marine environment where they spend much of the adult portion of their life cycle. Once reproductively mature, the fish seek to return to their juvenile stream, *i.e.,* the stream where the individual was stocked. The ultimate goal of the restoration effort is to create new, self-sustaining populations of these important fish. Not only do they have importance to the ecological functioning of the river, but they offer a sport fishing experience and are an important food source to larger marine fish (especially the herring, which are a main food source for stripped bass and blue fish along the coast).

Table 3.6-2
American Shad and Blueback Herring Transfers to the Ashuelot River (1998-2003)

Date	Agency	# Shad	# Herring	Town	Location
5/22/1998	NHFG	214	0	Swanzey	Above Homestead Woolen Mills Dam
5/29/1998	CONTE	70	50	Swanzey	Above Homestead Woolen Mills Dam
6/1/1998	CONTE	50	30	Winchester	Treatment Plant
6/3/1998	NHFG	248	0	Swanzey	Below Homestead Woolen Mills Dam
6/3/1998	CONTE	70	0	Swanzey	Above Homestead Woolen Mills Dam
5/24/1999	NHFG	228	0	Swanzey	Sawyers Crossing Bridge
5/25/1999	NHFG	246	0	Swanzey	Sawyers Crossing Bridge
6/7/1999	NHFG	128	0	Swanzey	Sawyers Crossing Bridge
6/8/1999	NHFG	78	0	Swanzey	Sawyers Crossing Bridge
5/31/2000	USFWS	0	95	Keene	American Legion
6/1/2000	NHFG	200	0	Swanzey	Sawyers Crossing Bridge
6/2/2000	NHFG	240	0	Swanzey	Sawyers Crossing Bridge
6/5/2000	NHFG	225	0	Swanzey	Sawyers Crossing Bridge
6/7/2000	USFWS	0	368	Keene	American Legion
6/12/2000	NHFG	220	0	Swanzey	Sawyers Crossing Bridge
6/13/2000	USFWS	0	389	Swanzey	Flatmill Roof Bridge, S.B. Ashuelot
6/5/2001	USFWS	0	568	Swanzey	Carlton Bridge, S.B. Ashuelot
6/7/2001	USFWS	0	786	Keene	American Legion
5/15/2002	USFWS	0	337	Keene	American Legion
5/22/2002	USFWS	0	330	Swanzey	Carlton Bridge, S.B. Ashuelot
6/4/2002	NHFG	188	0	Swanzey	Sawyers Crossing Bridge
6/5/2002	NHFG	247	0	Swanzey	Sawyers Crossing Bridge
6/9/2002	USGS	57	0	Swanzey	Tire Warehouse
6/10/2002	NHFG	195	0	Swanzey	Tire Warehouse
5/23/2003	NHFG	0	41	Swanzey	Sawyers Crossing Bridge
5/29/2003	NHFG	246	0	Swanzey	Sawyers Crossing Bridge
6/2/2003	NHFG	246	0	Swanzey	Sawyers Crossing Bridge
6/3/2003	NHFG	244	0	Swanzey	Sawyers Crossing Bridge
6/16/2003	NHFG	222	0	Swanzey	Sawyers Crossing Bridge
5/21/2004	NHFG	0	9	Swanzey	Sawyers Crossing Bridge
6/3/2004	NHFG	0	108	Swanzey	Sawyers Crossing Bridge
6/4/2004	NHFG	0	110	Swanzey	Sawyers Crossing Bridge
5/19/2004	NHFG	63	0	Swanzey	Sawyers Crossing Bridge
5/21/2004	NHFG	70	0	Swanzey	Sawyers Crossing Bridge
6/2/2004	NHFG	60	0	Swanzey	Sawyers Crossing Bridge
6/4/2004	NHFG	63	0	Swanzey	Sawyers Crossing Bridge
6/7/2004	NHFG	55	0	Swanzey	Sawyers Crossing Bridge
6/8/2004	NHFG	52	0	Swanzey	Sawyers Crossing Bridge
6/9/2004	NHFG	47	0	Swanzey	Sawyers Crossing Bridge
TOTALS		**4272**	**3221**		

Source: Gabe Gries, NHF&GD, personal communication

3.6.1.2 Fish Passage

As discussed above, the Homestead Dam bisects an important historic habitat for anadromous fish. It might naturally be assumed that removal of the Homestead Dam will improve fisheries habitat which is typically the case for dam removals. However, historical accounts mention a natural falls in the vicinity of West Swanzey, which has given rise to some concerns that removal of the dam may only expose a natural barrier to fish migration.

In some cases, dam removal expose natural channel features, such as rapids, that may, under certain hydraulic conditions, be difficult for certain species to pass upstream. For example, Atlantic salmon are capable of leaping falls as high as ten feet and so can navigate even fairly substantial cascades and falls. However, river herring and American shad do not leap but surmount obstacles by burst-swimming through chutes. Therefore, using the hydraulic modeling, the hydraulics of the exposed channel must be assessed relative to the swimming capabilities of targeted species. The hydraulics will be affected by the slope, geometry and flow ranges in the subject area during the migratory period for these species (spring and early summer).

Existing geotechnical data should reduce any concern that the dam is founded on a substantial falls. It must be understood that falls typically consist of bedrock or other consolidated deposits. The geotechnical borings completed by a contractor for the NH Department of Transportation in 2002 show little evidence of such a horizon. In only one of seven borings is a large boulder detected and bedrock was not discovered in any of the profiles. (See **Appendix C**). Interpretation of these boring logs does not reveal subsurface conditions that would be expected if the dam were located on top of historic falls.

Nevertheless, the expected slope of a new streambed in the vicinity of the dam would be on the order of 4%, which is a relatively high gradient and which might be expected to generate velocities that would impact fish passage. To examine this question, we used data from the HEC-RAS model reported in Section 3.2 together with published reports of fish swimming ability.

Atlantic salmon are known to be strong swimmers and are able to pass obstructions as high as ten feet through burst swimming and "jumping" over an obstruction. However, shad and herring are relatively weaker swimmers and therefore a closer analysis is warranted. First, note that these clupeids migrate upstream from June 1 to July 31 according to the Connecticut River Atlantic Salmon Commission's fish passage working group. Predicted maximum velocities for typical June and July flows are shown in **Table 3.6-3**.

Recently, Haro (2002) reviewed American shad swimming abilities in laboratory conditions and at natural high gradient reaches in New England rivers. This investigation found that passage of American shad through natural high gradient or high velocity reaches appears to be primarily dependent on water velocity and the distance to be traversed over which fish must be able to sustain high speed,

Table 3.6-3
Flow During Clupeid Upstream Migration

Month	Flow (cfs)	Velocity (ft/s)
June	400	2.5 to 7.5
July	180	1.5 to 6.0

Source: HEC-RAS Analysis, Appendix E

anaerobic swimming. Based on observations of 16 northern and mid-Atlantic rivers, Haro found that gradients of up to 2.7% over 100 to 200 m in length and rapids of class IV to V appear to be passable to shad. Shad have been observed to be able to swim in velocities exceeding 13 fps (Weaver 1965), although for relatively short distances. Haro's (2002) data found that nearly all shad tested were able to swim in excess of 30 meters at sustained velocities of more than 3 fps (roughly the length of a reconstructed channel if the Homestead Dam is to be removed).

Based on these observations, it is concluded that a restored channel would be passable to shad and other fish due to the fact that the stream gradient and velocities during the upstream migratory season will be within ranges commonly observed to be passable to clupeids in laboratory and natural settings. Nevertheless, the final design of any of the alternatives will need to carefully consider this issue in order to ensure effective fish passage.

3.6.2 Wetlands

3.6.2.1 Wetland Descriptions

According to the National Wetlands Inventory (NWI), the Ashuelot watershed contains 16,920 acres of wetlands representing 6.3% of the watershed. In actuality, there is almost certainly even more wetland acreage because many forested wetlands are not effectively mapped through NWI. There are three predominant NWI classified wetlands occurring in the Ashuelot basin:

- Palustrine wetlands include all nontidal wetlands dominated by trees, shrubs, emergent grasses and sedges, mosses or lichens in freshwaters;

- Lacustrine wetlands include wetlands and deepwater habitats associated with lakes, dammed river and stream channels, and large ponds (typically >20 acres), and lacking trees, shrubs, persistent emergents, emergent mosses or lichens with greater than 30% areal coverage; and

- Riverine wetlands include all wetlands and deepwater habitats contained in channels periodically or continuously containing flowing water or which form a connecting link between two bodies of standing water.

GIS data and field inspection of riparian wetlands by boat were performed in July and August 2004 to identify major wetland systems (>1 acre) along the Ashuelot River impoundment. **Table 3.6-4** provides a summary of wetlands systems that are adjacent to the river and **Figure 3.6-1** depicts their distribution along the river

Table 3.6-4
Wetlands Adjacent to the Ashuelot River, by Cowardin Classification

Wetland Type	Quantity	Size (acres)
PEM1/SS1E	1	14.06
PFO1A	2	10.66
PFO1C	11	181.38
PSS1/EM1C	1	1.34
PSS1E	3	26.01
PSS1F	1	4.17
PUBF	2	0.19
PUBH	1	0.42

Notes: "Wetland Type" follows Cowardin, et al. 1979. Data is from VHB GIS Analysis.

Because this reach of the Ashuelot is relative flat, aquatic bed communities are relatively common, particularly in meander areas. These aquatic beds are dominated by pickerelweed (*Pontederia cordata*) and arrowhead (*Sagittaria latifolia*), and also frequently include white pond lily (*Nymphaea odorata*), and other water lilies (*Nuphar* sp). An example of this type of wetland can be found on the inside of a meander bend approximately 1¼ mile upstream from the dam where velocities and depths are shallow.

A number of silver maple (*Acer saccharinum*) floodplain forests are also located along the bank of the river, including some forests which have been identified as "Exemplary Natural Communities" by the NH Natural Heritage Bureau. Informal discussions with the Bureau indicates that some level of impact to these floodplain communities is acceptable, given the overall benefit to the river created by restoring fish populations (Lionel Chute, personal communication, September 2004). More discussion of these forests is provided in Section 3.7.3 below.

Other floodplain forest wetlands are dominated by red maple (*Acer rubrum*) and American elm (*Ulmus americana*). Upland tree species adjacent to the wetland areas are dominated by eastern white pine (*Pinus strobus*), eastern red oak (*Quercus rubra*) and other typical northern hardwoods.

Shrub species along the edges include northern arrowwood (*Viburnum recognitum*), highbush blueberry (*Vaccinium corymbosum*), speckled alder (*Alnus rugosa*), and witch hazel (*Hamamelis virginiana*).

Ferns in the forested and scrub shrub wetland areas include cinnamon fern (*Osmunda cinnamomea*), royal fern (*Osmunda regalis*), and sensitive fern (*Onoclea sensibilis*). Other emergent plant species include bladder sedge (*Carex intumescens*), umbrella sedge (*Cyperus strigosus*), green bulrush (*Scirpus atrovirens*), soft rush (*Juncus effusus*), broad leaved cattail (*Typha latifolia*), skunk cabbage (*Symplocarpus foetidus*), and jewelweed (*Impatiens capensis*).

3.6.2.2 Potential Wetland Impacts

As discussed in Section 3.2, removal of the Homestead Dam will affect the hydrology of adjacent wetlands. Some of these wetland systems depend to some degree on the backwater conditions created by Homestead Dam. This relationship cannot be precisely defined however, and field observations found evidence that some supplemental surface flow enters the forested wetlands from the surrounding hillsides. However, this flow is likely inadequate to supply sufficient water to maintain the current hydrological regimes. To identify the potential extent of affected wetlands, a GIS analysis was completed.

The wetlands GIS analysis was performed using an estimate that backwater conditions currently caused by the Homestead Dam may influence wetlands to either side of the river for a distance of 300 feet in the vicinity of the dam. This distance gradually diminishes to no affect at the confluence of Swamp Ash Brook with the Ashuelot River. Therefore, a tapered buffer was created along the length of the Ashuelot, beginning at the Homestead Dam and extending upstream to Ash Swamp Brook using the above specified buffer distances. The buffer was overlaid onto NWI mapped wetlands. All NWI wetlands within the tapered buffer were identified as being potentially impacted by the removal of the Homestead Dam. These wetlands are listed by type in **Table 3.6-5.**

The vast majority of potentially affected wetland is classified as Palustrine forested (PFO) areas along the banks of the river, with scrub-shrub and aquatic bed/emergent marsh areas also present. In general, it can be predicted that removal of the Homestead Dam would shift wetland cover types such that aquatic bed communities would develop characteristics of emergent marsh systems. Scrub-shrub wetlands would likely acquire an overstory of silver and red maple, and understory species would shift to those characteristic of forested wetlands.

Table 3.6-5
Wetlands Potentially Affected by Dam Removal

Wetland Type	Quantity	Size (acres)
PEM1/SS1E	1	0.002
PEM1F	1	0.25
PFO1A	2	4.85
PFO1C	13	31.84
PSS1/EM1C	1	0.74
PSS1E	8	2.74
PSS1Eb	1	0.49
PSS1F	1	0.89
PUBF	2	0.19
PUBH	4	1.30
TOTAL	**34**	**43.3**

Source: VHB GIS wetlands analysis

Only at the very margins of the forested systems is there any potential loss of wetland acreage as marginal areas may be converted to upland. The exact quantity of affected wetland cannot be determined based on existing information. One should not interpret the data in **Table 3.6-5** to mean that there will be an overall loss of 43 acres of wetlands. Rather, the data show the approximate extent of wetlands where hydrological changes may induce observable plant community changes. Even these changes would occur over ecological time and would not be readily detectable for years to decades.

Loss of wetlands at the margin would likely be at least partially offset by the development of new riparian aquatic bed, emergent, and scrub-shrub systems within the reconfigured Ashuelot River channel. That is, with the drawdown of the impoundment, new surface area will be available to colonizing wetland plant species in areas currently submerged which would eventually form new wetland habitat.

3.6.3 Endangered Species Habitat

3.6.3.1 Dwarf Wedge Mussel

A single population of the federally–endangered dwarf wedge mussel (*Alasmidonta heterodon*) is known from the South Branch at its confluence with the mainstem Ashuelot River. The dwarf wedge mussel is a small freshwater mussel that is typically about 1.5 inches in length and is brown or yellowish-brown in color. Adult mussels are filter-feeders, feeding on algae and other small suspended particles. They spend most of their time buried almost completely in the bottom of streams and rivers. More information on threatened and endangered species can be found in **Appendix H**.

The life history of the dwarf wedge mussel depends on the tessellated darter, a native fish species, for dispersal of its progeny. Male dwarf wedge mussels release sperm into the water column during the mid-summer or fall. Females collect the sperm while siphoning water for food; the eggs are then fertilized and kept within the female until they are released the following spring. By then, each egg has developed into a parasitic larvae called a glochidium. After release from the female, the glochidium attaches itself to a fish with the aid of a small hook-like appendage. Mussel glochidia are generally species-specific and will only live if they find the correct host. With dwarf wedge mussels, the right hosts are small bottom-dwelling fish, the tessellated darter (*Etheostoma olmstedi*) and the mottled sculpin (*Cottus bairdi*). It appears that the glochidium receives little nutrition from the fish, but uses it only as a means of dispersal. After several weeks, the glochidium detaches itself from the unharmed fish and drops to the river bottom.

Typical habitat for this mussel includes running waters of all sizes, from small brooks to large rivers. Bottom substrates include silt, sand and gravel, which may be distributed in relatively small patches behind larger cobbles and boulders. The river velocity is usually slow to moderate. In the Ashuelot, all known populations of the mussel have been mapped, with one population known in the vicinity of the South Branch confluence with the mainstem Ashuelot.

The potential impact to the dwarf wedge mussel that might result from the removal of the Homestead Dam has already been studied by the US Fish and Wildlife Service, which found that the removal of the Homestead Dam is likely to benefit this federally-listed species. For example, the mussel's host fish species in the Ashuelot, the tesselated darter, is likely to find more favorable habitat and move upstream and downstream of the dam site. And, the removal of the dam will provide for a relatively higher velocity in the stream, which is a feature that the mussels prefer.

3.6.3.2 Floodplain Forests

Another concern is potential impact to patches of an unusual floodplain community located upstream. These floodplain forests are known as Silver Maple Floodplain Forests (*Acer saccharinum*), and have been identified by the NH Natural Heritage Inventory (now the NH Natural Heritage Bureau) as representing an Exemplary Natural Community. (See **Figure 3.6-1**.)Intact silver maple forests are somewhat uncommon in New Hampshire, in part because they were frequently cleared for farmland in the past. Natural Heritage Bureau records indicate that these floodplain forests are among "the best large patch floodplains on a medium size river in New Hampshire."

The potential impact to this community can be assessed based on the results of the hydrologic and hydraulic modeling. The results of the hydrogeological analysis suggest that there will be no effect on the floodplains forests from changes in groundwater levels in this area (which are predicted to be minimal).

However, floodplain forests are also driven by surface water flood events. It is known that floods between the 2-year event and the 50-year event are the most important in driving community composition in floodplain forests. Thus, by looking at the magnitude of change associated with these flow events in the vicinity of the floodplain forests we can develop some understanding of likely impacts. Although several forests are located along the Ashuelot within the Homestead Dam impoundment, it stands to reason that the forest closest to the dam itself would be most likely to be affected. This forest is located approximately 1 ½ miles upstream of the river. **Table 3.6-4** shows predicted water surface elevations under a dam in and dam out condition.

Table 3.6-6
Flood Elevations at Floodplain Forests

Flood Event	Flow (cfs)	WSE (fsl) Dam In	WSE (fsl) Dam Out	Change (ft)
2-year	2,940	460.88	459.48	1.40
10-year	4,630	463.11	462.08	1.03
50-year	6,190	464.81	464.04	0.77

Notes: fsl = feet above sea level
Data are from HEC-RAS Cross-section 51, the data nearest to the floodplain forest.

Based on the HEC-RAS results, it can be expected that flow events capable of flooding the forests will likely be less frequent and will be of shorter duration. For example, the ten-year flood elevation under the dam out condition (elev. 462) will still flood the majority of the adjacent forest, but the depth of this flooding is expected to be about one foot lower than with the dam in place. It is impossible to precisely quantify the effect that this might have on forest community dynamics. However, this magnitude of change seems unlikely to cause a sudden shift away from the silver maple community type. Rather, gradual changes in community composition may occur which would tend to allow plant typically occurring in drier sites to colonize the forest. As mentioned in Section 3.6-2 above, NH Natural Heritage Bureau biologists acknowledge that some community change may result from the dam removal, but find this effect acceptable given the overall benefit to the river through the restoration effort (Lionel Chute, personal communication, 2004.)

4

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Appendix A

Cultural Resource Reports (available upon request)

NHDHR Area Form
Phase 1A Archeological Study

Appendix B

Dam Inspection Report

Dam Inspection Photographs
NH DAMS 2002 Profile and Core Borings
Stability Calculations
1992 Dam Repair Photographs
References

Appendix C

Geotechnical Boring Logs

Appendix D

Cost Estimates Detail

Appendix E

HEC-RAS Data Summary

Appendix F

Sediment Sampling Results

Appendix G

Ashuelot River Clupeid Habitat Survey

Appendix H

Natural Heritage Inventory Data